JOURNAL

OF THE

AMERICAN WATER WORKS ASSOCIATION

VOL. 29 MARCH, 1937 No. 3

CONTENTS

Vacuum-Breaker Development for Back Siphonage Prevention. By F. M. Dawson and A. A. Kalinske..... 307

Improvements in Water Works Equipment and Materials By C. I. Dodd..... 322

New Water Supply for Little Rock, Arkansas. By Chester A. Smith..... 334

Water Hammer: Correcting an Unusual Problem. By Vance C. Lischer..... 343

The Disposal of Brines from Oil Drilling and Petroleum Operations. By Earnest Boyce..... 362

What Becomes of the Water after it Leaves the Pumping Station. By H. V. Pedersen..... 370

Activated Carbon—Its Manufacture and Storage. By J. P. Harris and E. A. Sigworth..... 377

Determination of the Density of Powdered Activated Carbon. By Edward S. Hopkins..... 390

Taste and Odor Problems. By William T. Bailey..... 392

Taste and Odor Problems at Kansas City, Kansas. By L. B. Mangun..... 399

Taste and Odor Problems at St. Joseph, Mo. By John P. Smouse..... 401

Tastes and Odors as Found in the Kaw River Water. By Frank E. Willey..... 406

Tentative Specifications for Gate Valves. Committee Report..... 408

New Publications..... 424

Abstracts..... 431

Coming Meetings..... v

Additions to Membership List..... vi

News of the Field..... 1

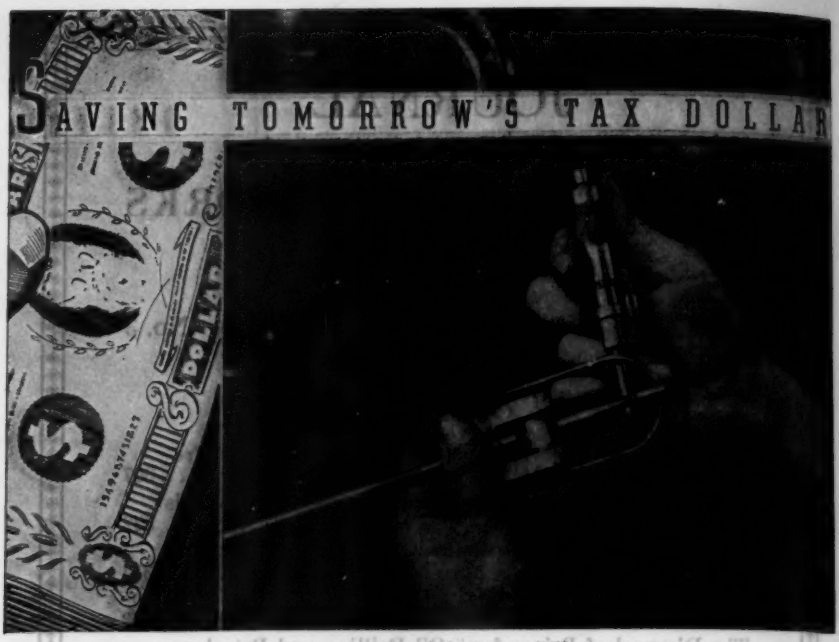
SECRETARY'S OFFICE, 29 WEST 39TH STREET, NEW YORK

All correspondence relating to the publication of papers should be addressed to Harry E. Jordan, Secretary, American Water Works Association, 29 West 39th Street, New York.

Because of the necessity for rigid economy, no reprints of articles will be furnished to contributors free of charge. Reprints may be purchased at the usual prices.

Subscription Price, $7.00 per annum

Single copies, to members—50 cents, to non-members—75 cents



Stoichometric accuracy is insisted upon for the control parts of W&T equipment. A W&T inspector checking a part for a Visible Vacuum Chlorinator

THE CHLORINATION DOLLAR

MUST BUY... ACCURATE CONTROL

TOO LITTLE chlorine in the treatment of water or sewage is useless,—too much, wasteful and unnecessary. To adequately perform its important task, chlorination must be accurately controlled.

Every part of the W&T Visible Vacuum Chlorinator is built to a painstaking standard — every detail of design, calibration and manufacturing is checked and rechecked to insure accuracy over years of service.

Visit a nearby installation. Notice the simplicity, ruggedness and fine workmanship. Note the absence of delicate parts or diaphragm valves — the ease of dismantling for cleaning or adjustment.

You'll no longer wonder why leading engineers and sanitarians everywhere endorse the W & T Visible Vacuum Chlorinator — nor why unfailing accuracy is the record of each installation.

Because of its low maintenance costs, tomorrow's taxpayers will also endorse your choice — if you specify Visible Vacuum Control.

Technical Publications 38, 157 and 158 are yours for the asking

WALLACE & TIERNAN CO. Inc.

Manufacturers of Chlorine and Ammonia Control Apparatus

NEWARK, NEW JERSEY • BRANCHES IN PRINCIPAL CITIES

THE ONLY SAFE WATER IS A STERILIZED WATER

JOURNAL
OF THE
AMERICAN WATER WORKS ASSOCIATION

The Association is not responsible, as a body, for the facts and opinions advanced in any of the papers or discussions published in its proceedings.
Discussion of all papers is invited

Vol. 29 MARCH, 1937 No. 3

VACUUM-BREAKER DEVELOPMENT FOR BACK-SIPHONAGE PREVENTION*

By F. M. Dawson[1] and A. A. Kalinske[2]
(University of Iowa, Iowa City, Iowa)

The problem of protection against back-siphonage and the potential dangers it presents to the purity of our water-supplies has reached a stage where it is necessary to review the whole subject in an unbiased, scientific, and comprehensive manner, in order to show what progress has been made, and whether that progress has been correct and sane. Since there is so much misunderstanding and gross misinformation on the whole subject of back-siphonage—among engineers, plumbers, manufacturers of plumbing equipment, public health officials, and others,—this paper will deal not only with the more simple aspects of siphons and siphonage but also with the more involved, though not necessarily difficult hydraulic, pneumatic, and practical problems that must be considered before protection, by whatever means, can be scientifically correct and practically and economically possible.

An attempt will be made to review the general means of protection against back-siphonage (with resultant water pollution) which have been suggested in the past. In addition the following will be discussed: (a) the conditions in the water-supply systems that vacuum-

*Presented before Missouri Valley Section, October, 1936.

[1]Dean of College of Engineering, University of Iowa.

[2]Plumbing Research Engineer, University of Iowa.

breaking devices must take care of, (b) the types of vacuum-breakers available, (c) the methods of testing of various protective devices in order to determine their effectiveness, and (d) what can be done to prevent back-siphonage to a large degree by correct and proper design and maintenance of water piping distributing systems.

DEFINITION OF TERMS

Before we go any further it would be proper to define certain terms in order to avoid confusion. As used in this paper back-siphonage will mean the process of flowing back of polluted or infected material or liquid into the water supply lines due to the formation of a less than atmospheric pressure in such supply lines. Absolute zero pressure, or practically so, will be referred to as a vacuum and any pressure greater than absolute zero, but less than atmospheric, as a partial vacuum. A vacuum-breaker is any device that relieves a vacuum or partial vacuum formed in the water-supply lines in such a manner as to prevent back-siphonage from submerged inlet fixtures or connections. Also those devices which some claim will entirely prevent the formation of any degree of partial vacuum in the water pipes will also be referred to as vacuum-breakers.

GENERAL METHODS OF PREVENTION OF BACK-SIPHONAGE

It is not necessary to go into any lengthy discussion of how partial vacuums are formed in water-supply lines; during the last few years numerous papers and bulletins on this subject have made clear to most everyone the many varied causes of partial vacuum formation in water lines. Undoubtedly the two most common general causes are shut-off of pressure with subsequent draining of the pipes, and usage of large amounts of water on the lower floors of a building when the supply pipes are too small. Among other causes may be mentioned the breaking of a water main or service line. A list of the more specific causes are: Fire pumps robbing the mains, heavy water demands during dry weather, turning off of pressure in building due to fixture stoppage, too small a riser in down-feed systems, and too small size of street mains to keep up pressure during peak loads.

There have been presented three general methods of protection of the drinking water-supply from pollution due to back-siphonage from submerged inlet fixtures containing contaminated matter: (1) raising of supply inlet sufficient distance above highest possible water level of fixture, (2) installation of separate supply to submerged inlet fixtures by having pure water discharge into an overhead tank which

would supply the unsafe fixtures, (3) installation of vacuum-breakers at all the unsafe fixtures and at other various points in the water-supply lines. The first is an adequate solution but unfortunately cannot be applied to many fixtures which inherently must have submerged inlets, such as flush valve toilet bowls, various hospital fixtures, many industrial processing tanks, various types of sinks, etc. Also, it must be considered that on some fixtures raising of the inlet is completely nullified by the user when some temporary connection, such as a hose on a sink or tub faucet, is made. This can only be taken care of by the third and by proper education of the public on the dangers of back-siphonage.

Some who feel that vacuum-breaker protection is quite inadequate say that all unsafe fixtures, referring particularly to closet bowls, should be installed on a separate system fed by an overhead tank supplied with water with an above overflow float valve. Besides the economic factor and the fact that in small buildings the head would not be high enough to operate most flush valves, the main argument against such a solution to the back-siphonage problem is that one evil is substituted for another which is perhaps worse. Every building would then virtually have two water-supply systems, one safe and another whose purity could not be depended on. The chances of the piping being confused and installation of pure water supply fixtures on the gravity tank system or cross-connecting the two systems would be very great. The dual system would complicate the plumbing when the trend has been to simplification. It took generations of education to get rid of dual supplies. Let us not go backwards and have the same troubles over again. Those who advocate such a solution forget the fact that there are many fixtures, especially in hospitals, to which a pure water supply is essential, and still the fixture can have contaminated water in it and thus permit back-siphonage. A few such fixtures are: therapeutic baths, aspirators, sterilizers, autopsy tables, bidets, various sinks, etc. Obviously the only protection for such fixtures is the vacuum-breaker method, and if it is admitted that a vacuum-breaker is effective on one fixture, it must necessarily be adequate for another.

From the foregoing discussion it is obvious that some device installed on a fixture, or perhaps on the piping near each fixture, is the method that affords a simple, economical, and most applicable solution to the problem of back-siphonage. Why the installation of protective devices on the individual fixture is a much more ideal solution than on the water piping will be discussed later.

HYDRAULICS AND PNEUMATICS OF PARTIAL VACUUM FORMATION IN WATER SUPPLY LINES

Most protective devices which are known as vacuum-breakers, prevent back-siphonage by breaking the vacuum or partial vacuum formed or forming in the water-supply lines, in such a manner as to preclude any possibility of raising any polluted water from the fix-

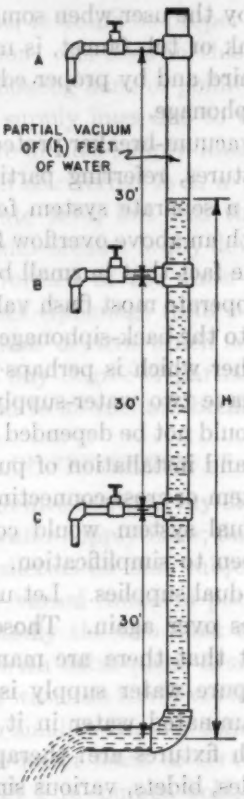


FIG. 1. VACUUM FORMATION ABOVE DROPPING WATER COLUMN

ture on which, or near which, they are installed. In any installation the two most unfavorable negative pressure formation conditions are: (1) the vacuum-breaker is called upon to supply air as soon as the partial vacuum starts forming and the water column is receding, (2) when a volume of water piping has been emptied and a practically perfect vacuum exists, and then a valve or fixture is opened and air rushes in through the breaker at a high rate at first and then at a

gradually decreasing rate till the vacuum is dissipated. In the first case the partial vacuum may not be so very high, but it persists for a relatively long time. In the second case the vacuum at the start is very high, but decreases quite rapidly to zero.

Let us analyze these two conditions in order to determine exactly what degree of vacuum will be formed in the water-supply lines, and what volume of air will flow through any known opening in a vacuum-breaker. This will enable the correct design of a vacuum-breaker to take care of any pre-determined maximum conditions, and also point out a means of making a practical test on a vacuum-breaker. Figure 1 is a diagrammatic sketch of a water riser in a multiple story building showing openings at three points. For a maximum condition, suppose there is a break near the bottom of the riser so that a full size opening exists for the water to flow out. If as soon as a partial vacuum starts forming one vacuum-breaker opens and supplies air, say the one on floor (A), what is the degree of vacuum above the water? Knowing the degree of vacuum the rate of air flow can be tested for or roughly calculated for any given vacuum-breaker. The head producing flow of water is equal to $(H_w - H_a + H)$ and the rate of water flow in c.f.s. equals:

$$Q_w = K\left[\frac{D^5}{L}(H_w - H_a + H)\right]^{\frac{1}{2}} \qquad (1)$$

where

$$K = \left(\frac{2g\pi^2}{16\,f}\right)^{\frac{1}{2}}$$

f = pipe friction coefficient.

L = length of pipe, for maximum condition equals H_w

H_a = atmospheric pressure in feet of water, 33′.

H = absolute pressure above the water in riser, and equals $(H_a - h)$.

The flow of air through any opening cannot be expressed in terms of ordinary hydraulic equations because for large pressure differences the flow of air must be analyzed by the laws of adiabatic expansion of gases. By thermodynamic principles it can be shown that the rate of air flow through any opening in lbs. per second is:

$$Q_a = 190\,CA\sqrt{\left(\frac{P}{P_a}\right)^{2/k} - \left(\frac{P}{P_a}\right)^{\frac{k+1}{k}}} \qquad (2)$$

P = absolute discharge pressure in lbs. per sq. ft.
P_a = atmospheric pressure in lbs. per sq. ft.
C = friction coefficient
k = 1.4 for air.
A = area of air inlet in sq. ft.
(190 is obtained by combination of various known quantities for air under 70°F. and 14.5 lbs. per sq. inch pressure.)

It should be pointed out that the value Q_a becomes a maximum when $\frac{P}{P_a} = .53$. This means that the flow-rate of air in lbs. will not increase after the absolute pressure in a water riser reaches .53 × 33 or 17.5 of water which is equal to $(33 - 17.5)\frac{12}{13.6}$ or about 14 inches of mercury vacuum. In other words, as far as air flow through any opening or vacuum-breaker is concerned any vacuum greater than 14 inches of mercury will subject it to the maximum condition.

With water dropping in a riser and air entering through some opening the vacuum that is formed above water can be obtained by equating the rate of air and water flow:

Changing Q_a to volume units of air under pressure conditions in the riser by dividing Q_a, which is in lbs. per second, by the unit weight of air under pressure P we obtain,

$$Q_a' = \frac{190\ CA}{w_a}\left(\frac{P_a}{P}\right)^{1/k}\sqrt{\left(\frac{P}{P_a}\right)^{2/k} - \left(\frac{P}{P_a}\right)^{\frac{k+1}{k}}}$$

$$= \frac{190\ CA}{w_a}\sqrt{1 - \left(\frac{P}{P_a}\right)^{\frac{k-1}{k}}}$$

It must be remembered that the air flow expressions are not valid for P less than .53 P_a, after P reaches .53 P_a, Q_a remains constant, and Q_a' can be expressed in volume units of the pressure existing in the riser, if it is less than .53 P_a, by solving for Q_a' with $P = .53\ P_a$, and multiplying the result by $\left(\frac{.53\ P_a}{P_a}\right)^{1/k}$.

Since $Q_a' = Q_w$, there is obtained:

$$M(H_w - H_a + H) = 1 - \left(\frac{H}{H_a}\right)^{1 - \frac{1}{k}}$$

Note that: $$\frac{P}{P_a} = \frac{H}{H_a}$$

$$M = \frac{w_a^2\, k^2\, D^5}{190^2\, LC^2\, A^2}$$

Taking k at 1.4 for air and $H_a = 33'$ of water we obtain:

$$H^{.29} + 2.76\,MH = 2.76\,[1 - M\,(H_w - 33)] \qquad (3)$$

(Values of H are to be obtained by trial.)

If the value of H is less than .53 H_a then the expression above is not valid and H should be solved for by equating the value of Q_a' mentioned above to Q_w.

Example:

If the riser pictured in figure 1 is 3 inches in diameter and the air-opening on the vacuum-breaker is $\frac{3}{4}$ inch (having a coefficient of about .50) it is required to find the vacuum in the riser when the water column is at A, B, and C. Solving the above equation gives values of partial vacuum at the points mentioned of 7.0, 6.6, and 6.0 inches of mercury. Thus it is seen that a fairly constant vacuum is maintained inside the riser as the water is dropping and air entering through the vacuum-breaker. Of course, the larger the air openings are made in the vacuum-breaker, the less will be the partial vacuum; but the pressure in the riser will never be atmospheric, as some difference in pressure is necessary to make air flow, no matter how large the air opening.

The above condition is the only way a fairly constant partial vacuum can be maintained in any water-supply piping system when air is flowing in through a vacuum-breaker. Therefore, to test any vacuum-breaker to see if it will prevent siphonage under such conditions, all that remains to be done is to set what is considered a maximum practical condition as far as height and size of riser is concerned, and then test the breaker by maintaining a constant partial vacuum of such magnitude as would occur in the selected water-supply riser. Such a partial vacuum could be maintained by means of a vacuum pump. It must be remembered, however, that on any given installation if a partial vacuum forms there will be undoubtedly more than one vacuum-breaker to supply air.

The above described condition is not the most severe that any vacuum-breaker must withstand. It is quite possible that a water riser, (one that is over 33 feet), may empty down to 33 feet without any vacuum-breaker coming into action, either because a toilet flush valve remains closed for some reason or other, and no other fixture is opened. Supposing *after* this so-called "volume of perfect vacuum" or void has been formed, a flush valve should suddenly snap open or any other submerged inlet fixture having a vacuum-breaker be opened, then because of the high vacuum existing in the pipes, air

would flow in at a very high rate and the rate would remain constant till the vacuum was reduced to 14" of mercury. Below the 14" vacuum the rate of air flow would drop off rapidly. It might be of interest to know how fast any given "volume of vacuum" would be dissipated through a given size air opening. The term "volume of vacuum" as used here means the void space under a certain negative pressure.

The time required for the vacuum in a volume (V) to be reduced down to the critical value of 14" of mercury, or in absolute units .53 of atmospheric pressure, can be readily obtained since Q_a (lbs./sec.) is constant.

Therefore t in seconds:

$$= \frac{V(w_c - w_o)}{Q_a} \tag{4}$$

Where:

w_c = weight of air at pressure of .53 P_a.
w_o = weight of air at initial pressure.
Q_a = max. rate of air flow in lbs./sec.
V = cubic feet of volume.

The time required for dissipation of a vacuum above the critical pressure (P_c) is obtained by evaluating the following integral:

$$t = \int_{P_c}^{P} \frac{V dw}{Q_a}$$

Letting $w = w_a \left(\frac{P}{P_a}\right)^{1/k}$ and substituting the expression for Q_a, the integral evaluates to:

$$t = \frac{2Vw_a}{190\,CA(k-1)} \log_e \left[\frac{\left(\frac{P_a}{P_c}\right)^{n/2} + \sqrt{\left(\frac{P_a}{P_c}\right)^{n} - 1}}{\left(\frac{P_a}{P}\right)^{n/2} + \sqrt{\left(\frac{P_a}{P}\right)^{n} - 1}} \right] \tag{5}$$

$$n = \left(1 - \frac{1}{k}\right)$$

where k for air = 1.4.

The expression before the log term reduces to $\frac{.002V}{CA}$.

It takes about 8.5 seconds to reduce a vacuum of 29 inches to 14 inches of mercury in a 100 gallon volume with air flowing in through a $\frac{3}{4}$ inch opening having a value of C equal to .50. It takes about 12.5 seconds to dissipate the vacuum down to 5″ of mercury. Remember that a 100 feet of 2-inch standard iron pipe has a volume of 17.5 gallons and 100 feet of 4-inch a volume of 66 gallons. Thus when any fixture or vacuum-breaking device is tested under a sudden application of a high vacuum, the volume of the vacuum should be of practical size and not out of line with what could occur actually.

The conditions described represent the severest tests any vacuum-breaker, depending for its effectiveness on the flow of air into the pipes, can encounter. Though any vacuum above 14″ of mercury will cause the maximum rate of air flow, certain flush valves and other fixtures should be tested with as high a vacuum as can be obtained with water, because the opening up of the flush valve may depend on the magnitude of the force exerted. There are other partial vacuum conditions that occur in water pipes, but they do not require much air for their relief. For instance, wherever air flows into a pipe which still has water, the rate of air flow will never be high, due to the resistance of the water. Also, air will not flow in at a very high rate due to a partial vacuum in a down-flow riser flowing full of water, or below the top of a dropping water column in an up-feed system.

This discussion indicates something of the nature of vacuum formation in water pipes, and will help to point out why some vacuum-breakers are effective and some are not. The mathematical analyses presented may seem unduly complicated, however from them can be obtained an idea of the magnitude, volume, and time of persistence of vacuum conditions in water supply lines.

EARLY AND PRESENT TYPES OF VACUUM-BREAKERS

The first type of vacuum-breakers for prevention of back-siphonage had a $\frac{1}{16}$ or $\frac{1}{8}$ inch size air opening which was closed by a ball-check to prevent spitting of water (Figure 2—Item A). These devices could be placed on either the pressure or discharge side of the control valve to the fixture. Obviously, such a device, or in fact any vacuum-breaking device whatsoever, cannot be depended on if it is placed on the pressure side of the control valve where the air openings are always kept closed. There is no assurance that the openings will open should a vacuum occur after some sort of check

had kept them closed for months or perhaps years. However the main fault of these early breakers was, of course, their size. The amount of air they allowed to pass through did not prevent the formation of a partial vacuum in the fixture connection, with the result that polluted water could rise up from the fixture, mix with the air coming from the vacuum-breaker, and pass back into the lines.

The next step was the development of the Venturi vacuum-breaker which had two or three $\frac{1}{8}$ inch air holes (Figure 2—Item B). These were an improvement, as they had nothing that could get out of

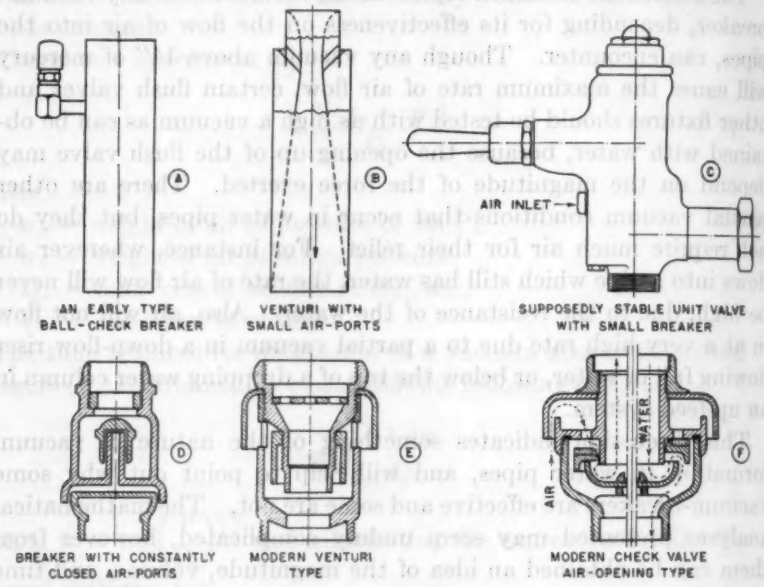


FIG. 2. VACUUM BREAKER DEVELOPMENT FOR CLOSET BOWL FLUSH VALVES; "E" AND "F" ARE DEPENDABLE

order and supplied much more air for vacuum relief. However, they caused a noisier flow of water because of the restricted opening, and also, if not designed correctly for a particular fixture, they would "spit" water. Other developments included small air openings placed on the flush valve and various types of vacuum-breakers having constantly closed air-ports (Figure 2—Items C and D). Neither were in any way adequate or dependable.

The next developments which occurred practically simultaneously were the improvement of the Venturi vacuum-breaker and the de-

velopment of the combination "check-valve-air-opening" vacuum-breaker (Figure 2—Items E and F). The latter type was developed primarily because of the many difficult problems presented by the Venturi breaker, such as excessive size of air openings, noise, and necessity of different types and sizes for different fixtures and water pressure conditions. The "check-valve-air-opening" type, of which there are three or four different makes on the market today, work on the principle that the main water passage is partially closed by a loose check valve when the slightest vacuum occurs (or when flow ceases), and therefore it will not be necessary to provide such tremendously large air openings. Also, by special design a check closes the air openings when water flows, thus preventing spitting of water. These breakers *must be placed on the discharge side of the control valves*, and all the movable parts of the breaker must be actuated each time the fixture is used. The air openings are always open, except during the time water is flowing under pressure to the fixture. They open again immediately that there is the slightest vacuum and also when the flow of water stops. In fact the above is a list of necessary requirements for the "check-valve-air-opening" vacuum-breaker.

Recently the Venturi type of breaker has been improved considerably and is a device that can be used with fair success if a little care is employed in installing the correct type (Figure 2—Item E). In fact, one manufacturer has incorporated this type of breaker into a closet bowl, thus eliminating the necessity of matching the breaker with type of bowl being used.

Neither the Venturi nor the "check-valve-air-opening" types of vacuum-breakers can be considered as perfect, they undoubtedly can and will be improved in the future. In general vacuum-breakers of the above two types will prevent back-siphonage under any condition if the air passages are large enough and otherwise designed correctly. These items must be determined by test.

OTHER BACK-SIPHONAGE PREVENTION DEVICES

Another much talked about siphonage preventing device for flush valve fixtures, is the "stable unit" flush valve (Figure 2—Item C). The "stable unit" flush valve is a flush valve whose piston unit cannot be pulled open by ordinary application of high or low partial vacuums. Some have maintained that such a flush valve presents a positive protection against the occurrence of a full opening in the water-supply line, and thus only a very small vacuum-breaker is

required. However, it is quite apparent to anyone giving the matter thought and after a study and testing of such "stable unit" valves, that such items as wear, corrosion, dirt and rust particles, sticking of certain parts of the valve due to long periods of non-use, and many other factors render such a stable valve only relatively stable. The "stable unit" is a very fine additive protective feature on any flush-valve installation, but it cannot be depended on, and a vacuum-breaker of the proper size must be used along with it.

Another means of protection against back-siphonage that has been urged by some engineers is the installation of vacuum-breakers on tops of risers and other high points in the piping systems. The chief objection to such a method is that the vacuum-breakers would always be under water pressure, and therefore absolutely no dependence can be placed on their functioning when needed. Another item is that it is very difficult to install a sufficient number of these devices so that no partial vacuum will occur in the water pipes. Since the fixtures have no protection, no degree of vacuum can be allowed to occur for even a fraction of a second. This, as our discussion on the hydraulics and pneumatics of vacuum formation pointed out, is entirely impossible. The installation of vacuum-breakers on the top of risers and various other high points will not prevent the momentary formation of small partial vacuums in some of the horizontal branches.

In addition, partial vacuums frequently form in horizontal lines due to negative "water-hammer" pressure waves; which could not be prevented by the water-riser vacuum-breakers. Everything considered, the only positive protection from back-siphonage to date, is the correct installation of proper vacuum-breakers on all fixtures presenting potential water pollution dangers.

At this point should be mentioned the controversy in progress among plumbers and engineers as to how high should the water-supply inlet be raised above the maximum water level for any fixture. No definite distance can be set that will apply to all fixtures; such important variables as type and condition of end of inlet, diameter of inlet, type of liquid and condition of surface, all greatly effect the height to which a liquid will jump up when air is being pulled into the inlet by vacuum conditions in the pipes. All that can be done is for fixture designers to test any particular fixture under vacuum conditions that can and will occur in the water pipes, operating the fixture as it would be operated in an actual installation,

and using a liquid and surface condition that goes with that type of fixture. The type of valve connected to the inlet under test is important, because whether it can be opened quickly or not is a very important factor. Such a test will give definite information as to the maximum height the water can be raised off the surface.

TESTING OF VACUUM-BREAKERS

Many individuals interested only in demonstrating the terrifying aspects of back-siphonage and how it can cause pollution of water, have discarded all fact and reason, and proceeded to demonstrate and test vacuum-breakers under conditions inconceivably fantastic. After all, a vacuum-breaker is built for the special purpose of supplying air to water pipes when a partial vacuum occurs due to certain definite hydraulic phenomena, and is not built to relieve vacuums in immense tanks from which the air is being exhausted by pumps connected to 10, 15, and even 25 horse-power motors.

Any test of vacuum-breakers must be based on what can and does occur in water-piping systems. Once this is done and the maximum conditions that any vacuum-breaking device should take care of are set, providing of course for a proper factor of safety, then the testing and approving of these devices will be put on a sane basis and the present chaotic condition eliminated.

The hydraulic and pneumatic conditions relating to the formation of vacuums in water supply pipes have been discussed and analyzed. Now there will be pointed out certain definite factors and procedures to consider in the testing of vacuum-breaking devices:

(1) If a large number of vacuum-breaking devices are to be installed on fixtures in certain size buildings having practically similar water-piping systems it would be worthwhile to study the size, volume, and other installation features of the piping in such buildings, and a test devised duplicating the maximum vacuum conditions that can occur. "Vacuum conditions" refer to such items as magnitude, volume, and persistence of the vacuum. These items could be immediately determined if we would set a certain definite size and height of water riser that any breaker would have to prevent siphonage into. Also, if there is a possibility of the water mains being drained, as may happen quite readily in certain hilly cities, that factor must be taken into account. It should also be remembered that the greater number of vacuum-breakers installed in any building the easier it will be for any one breaker to prevent siphonage

from its particular fixture. There is every reason in the world why a different size and type of vacuum-breaker should be required in a 2-story residence than in a 10-story apartment house or 50-story skyscraper.

(2) Figure 3 shows a test installation for a toilet flush-valve vacuum breaker. It is important that the breaker be installed at some definite pre-determined elevation above the maximum possible water level, and then a limit set as to how high the polluted water can rise in the supply connection. For instance, it might be stated that with the breaker installed 3 inches above the fixture the device should prevent polluted water from rising above one inch under the maximum vacuum conditions. It is, of course, assumed that polluted water can rise to the top of the fixture itself.

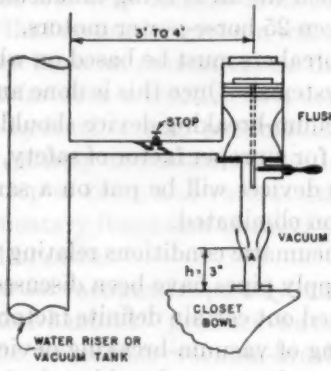


FIG. 3. FLUSH VALVE AND AIR-BREAK INSTALLATION FOR TEST

(3) All mechanical vacuum-breakers, and vacuum-breakers built integrally with the flushing device, should be subjected to a test where the partial vacuum is varied considerably in a short interval of time, and also having the pressure change rapidly from positive to negative and vice versa. Though such a test is of not great consequence for the non-mechanical breakers which take care of a full line opening, certain mechanical breakers and devices depending for their proper functioning on "stable unit" flush valves have failed in such a test. It is quite probable that such pressure variations may occur in the taller buildings and buildings using large volumes of water at odd moments.

CONCLUSION

It is hard to imagine how all of the potential dangers of back-siphonage, especially the more remote ones, will ever be eliminated unless all persons having anything to do with the designing, installing, maintaining, and using of water-supply and plumbing systems become thoroughly acquainted and convinced of the dangers of water-pollution through back-siphonage. This obviously entails considerable education and preaching—a large burden of which must be borne by men in the water-supply and plumbing business. For instance, who knows when some uninformed user will attach a hose to an otherwise safe raised inlet fixture. Who can entirely prevent an owner, or a handyman he employs in his home or factory, from making changes in the water piping system and perhaps creating water pollution dangers.

It is the business of public and private water departments or companies to deliver pure and wholesome water at all times to their consumers, and therefore the problem of back-siphonage and resultant pollution of that water, the commodity they are selling, becomes their concern. Rules and regulations will and can do much, but education must come first. Intelligent piping design by engineers and the proper installation of the piping and pumping appurtenances by qualified and trained plumbers, are the greatest factors in elimination of back-siphonage dangers. Thorough understanding of the more simple causes of back-siphonage by all who have anything to do with piping and plumbing systems will make the efforts put forth in proper design and installation not in vain.

The causes and solutions to the problems of back-siphonage and cross-connections are known. Potential dangers due to any possible back-siphonage can be easily, cheaply, and permanently eliminated, if science and sanity are substituted for ignorance and commercialism.

IMPROVEMENTS IN WATER WORKS EQUIPMENT AND MATERIALS*

By C. I. Dodd

(*Engineer, Black & Veatch, Kansas City, Mo.*)

Improvements in materials and equipment used in water works practice may be placed in three general classifications.

First is refinement in design, shop practice and the use of better materials; examples of which are steam generating equipment, pumping equipment, pipe, valves, hydrants, elevated tanks, centrifugally cast iron pipe, copper tubing, stainless steel and protective coatings.

Second is the application of standardized devices and equipment to water works service, in some cases to functions somewhat foreign to their original uses. This classification would include the great increase in the use of electrical power in performing functions previously handled by water motors, gravity and by manual operations; the increased motorization of valves, sluice gates, cranes and hoists; the use of conveying equipment, both mechanical and pneumatic, in connection with overhead storage bins for the handling and storage of chemicals in bulk, instead of packaged chemicals handled with hand trucks and cumbersome elevators; the application of float switches, pressure switches, electrical relays, contactors, solenoids and other mechanical and magnetic devices to automatic control and operation of water treatment and pumping plant equipment; the use of compressed air for aeration and rapid mixing; the application of activated carbon and ammonia to water purification; and certain types of chemical mixing and feeding equipment, although chemical equipment borrowed from the chemical industry has, in some cases, been considerably modified to meet the special conditions of water treatment. There are several items of equipment which could be placed in a sub-division of this second classification, although perhaps they should be given a classification of their own. I am referring to equipment either developed entirely for sewage treatment work and

*Presented before the Missouri Valley Section, October, 1936.

adapted to water treatment, or in some cases borrowed from other industries and applied to both sewage and water treatment at about the same time. Included in this group will be found grit removal equipment, sludge pumps, porous air diffusor plates for compressed air aeration and certain types of mechanical sludge collection equipment, the old types of which were adaptations of similar equipment used in the mining industry in the preparation and segregation of ores. The great need to reduce the cost of maintenance and repairs has stimulated the development of better protective coatings, involving considerable research work and service testing of paints made from synthetic gums and resins; and the use of copper, aluminum and the corrosion resisting alloys, although not to the limit of ultimate economy on account of the quite generally applied limitation of first cost.

As an example of the adaptation of an existing device to serve a purpose different than its original function, I wish to describe two such adaptations which our firm has made in our design work. We do not claim that such adaptations are original with us, but no manufacturer had what we wanted and we decided that the only way to get it would be to specify something special and see what happened. In each case the manufacturers gave us what was wanted without question.

We had been specifying wash water rate controllers on all filter plants. This, of course, is the ideal arrangement, and is necessary if the wash water storage is at too high an elevation. Rate controllers are expensive, however, and in many small plants the cost of wash water rate controllers is hard to justify, especially if, as is usual, it is necessary to keep costs to an absolute minimum. We decided that the head loss through a filter during backwashing would be a fairly accurate measure of the rate of wash, at least over a period of time during which there would be no material change in the condition of the filter underdrains and filter media, and that the loss of head gauge, if of the mercury or weighted pendulum type, could be easily adapted to indicate positive as well as negative pressures, and by connecting one side of the gauge to the center line of the effluent manifold and the other side to the filter box at the exact elevation of the top of the wash troughs, and having the gauge indicator calibrated each way from zero, that the gauge would indicate loss of head on one side of the zero position and the friction and velocity head losses through the filter on the other side of the zero position

of the indicator pointer, inasmuch as the static pressure in the gauge piping would exactly balance that in the filter. Each gauge would be calibrated by trial to determine the head loss at the desired rate of filter wash and an adjustable red pointer on the gauge dial set at that position. In washing the filter, the operator would bring the gauge pointer to coincidence with the red pointer and hold it there during the wash. It is necessary that the calibration be checked at intervals to correct for changed conditions in the filter. To protect the filter from damage through accident or carelessness in washing, we always try to fix the height and depth of the wash water storage so that it will be impossible to obtain a rate of wash in excess of a thirty-six inch per minute rise in the filter, and at the same time provide a minimum of not less than twenty inches when the tank is empty. If a wash water pump is used, the head-discharge characteristic of the pump is limited in the same manner.

Another adaptation we have made is to apply the standard mercury float-well type of rate of flow meter register to serve as a depth gauge or weir meter. In order to keep an automatic record of filter washes in small plants, and at the same time to provide a means for checking the amount of wash water used without the expense of metering the supply, it has been our practice to install a recording depth gauge to record the depth of water in the wash water storage. Believing that simplicity is always desirable, and there is no good excuse for using electrical instruments where mechanical gauges can be used, we have specified that this depth gauge was to be of the mercury manometer float operated type. Since the wash water tank head range is usually low (we try to keep it within twelve feet) a standard 120″ differential rate of flow meter can usually be used as a depth gauge with no more change than the preparation of special charts reading in feet instead of rate of flow. The same thing can be done in the application of a flow meter register to indicate, record and totalize the flow over a weir. In this case, the instrument must be designed to register the rate of flow as determined by the depth of flow over a weir instead of by the pressure differential through an orifice or Venturi tube.

The third general classification includes the development of new equipment and devices for specific purposes in the handling and treatment of water alone, such as rapid sand filters, filter effluent rate controllers, loss of head gauges and automatic lime slaking equipment.

Among the more recent additions to this classification are: mechanical aerators; chemical mixing and agitating equipment, both slow and fast; the development of basin agitating equipment from small individual basin units to assemblies operating in large basins; carbon dioxide generating equipment; new filter underdrain systems free from corrodible materials; reducing or eliminating entirely filter gravel for diffusion of the back wash; automatic and semi-automatic control of filter valve operation; and the development of new synthetic zeolites for water softening.

PUMP IMPROVEMENTS

The usual practice of evaluating water works pump bids on an efficiency basis has kept the pump manufacturers busy striving to outdo their competitors in the matter of high pump efficiencies. As a result, much improvement has been made in centrifugal pump performance, especially in the field of single stage, high speed pumps and deep well turbine pumps. Earlier designs of high speed single stage pumps were not always satisfactory due to bearing troubles, but the modern high grade pump seems to operate at 3500 R.P.M. in as dependable a manner as the older, slower speed, multi-stage type units, and at much better efficiencies. Higher efficiencies have been developed in deep well turbine pumps, and in many instances pumps of this type will operate at as high or higher efficiencies as pumps with volute type impellers under equivalent head and discharge conditions. This improvement in efficiencies has been obtained partly through the use of highly polished surfaces of impellers and in casings, and in some cases by covering the water passages with vitreous enamel.

A new pump of the deep well turbine type has been developed to eliminate trouble due to shaft elongation in deep settings and high heads. This is done by dividing the bowl assembly into two parts, with one-half opposing the other half to provide hydraulic balance in the pump. Standard deep well turbine pumps can be made to meet a wide range of heads and capacities by varying the number and curvature of the impeller vanes.

In order to handle very large capacities from wells of relatively low bore, so-called "axial flow" pumps, using impellers of the propeller type have been developed and seem to fit certain conditions very well on account of their inherent low-head characteristic. A type of pump developed for use in irrigation work, making use of a screw

propeller operating in a cylindrical casing, will handle large quantities of water at very low heads and because of the large clearances through the pump may find use in the water works field for raw water service, or for handling lime sludge. The efficiency of this type of pump is still relatively low.

Because of the ease of installation, high efficiency, and small space requirements, deep well turbine pumps are very satisfactory for low-head secondary pumping around a filter plant, such as pumping filtered water into filtered water storage where the filters are too low for gravity drainage. They may also be conveniently installed in river or lake intakes, but should be protected by fine screens if used for this service.

Turbine driven centrifugal pumps have almost entirely displaced plunger pumps on steam plants, on account of the saving in space requirements and in cost of the equipment and building construction. In many instances, the capacity of a steam pumping plant has been greatly increased with no change in the building housing the pumps by replacing crank and fly wheel pumps with steam turbine driven centrifugal units.

PIPE

The principal change in water works pipe line and distribution system construction during the last decade has been in the more or less general abandonment of the old A. W. W. A. standard cast iron pipe, in favor of the use of lighter weight pipe made from cast iron possessing relatively high tensile strength. This change may be partly due to the development by pipe manufacturers of improved manufacturing methods and facilities, but probably due more to the fact that the lighter weight pipe can be sold at a cheaper price, especially where freight constitutes a large proportion of the selling price. In absence of any other specification covering pipe of this type, outside of manufacturer's specifications which were not standardized, the pipe manufacturers have largely adapted their product to meet the requirements of the specification for bell and spigot cast iron water pipe designated as Federal Specification WW-P-421, which is used by many engineers at the present time. This specification admits DeLavaud and Mono-cast centrifugally cast pipe and McWane horizontally cast pipe. Federal Specification WW-P-423 covers cast iron pipe of the bolted joint type. Flanged pipe and all special castings for water pipe are made by most pipe manufacturers

under the old A. W. W. A. standard specifications, although lighter weight fittings under the Federal Specification strength classifications are also available.

Concrete pressure pipe has been improved in quality along with better knowledge of concrete manufacture in general. Concrete pressure pipes made by the centrifugal process are available for low or moderate heads, but pipe for high heads are generally provided with a welded steel cylinder cast in the body of the pipe barrel, which prevents all seepage or leakage through the concrete. Joints sealed with rubber gaskets are now being used, which provides more joint flexibility, both in installation and for pipe line settlement, than the rigid type previously used in concrete pipe.

Apparently little has been done in the United States relative to the process of pre-stressing concrete pipe with bands or spiral wrapping under predetermined stresses along the line of the "Hewitt" system of concrete construction as used in reservoirs, tanks and stand pipes. The theory is good but it may be that manufacturing difficulties have retarded the development of this type of construction.

Asbestos-cement pipe is now available for water pressure service, and may in time become a prominent factor in the pipe business.

The development of the process of spinning enamel linings in steel pipe, together with the excellent outer coatings and wrappings developed for the oil and gas industry, affords better protection for steel pipe and may tend to increase the confidence of engineers in specifying steel pipe for the transmission and distribution of water on an equal basis with cast iron pipe.

One noticeable development in the past decade has been the widespread use of copper tubing for water service connections. Combining the flexibility and ductility of lead, which is of great value in prevention of line breakage due to sub-grade settlement, shocks, vibration, and freezing, with an installed cost comparable with that of wrought iron, copper water tube has met with almost universal approval and is rapidly displacing other kinds of pipe for this service.

VALVES AND HYDRANTS

There has been little change in the design of water works valves. One of the more important developments is the increased use of square bottom gate valves. Ordinary gate valves are unsatisfactory for throttling service or use in vertical lines due to the tendency for the gate to tilt into the down-stream port opening and, since they are

more or less balanced on a two point support, have a decided tendency to flutter or vibrate when used in a throttling position. When the gate is closed against much of a differential head this tilting action will also tend to cause the gate to cut into the gate seat. Sliding shoes operating on guides or tracks in square bottom gates provide a four point support.

The automobile and truck hazard has brought about radical improvement in the design of fire hydrants. The new type of hydrant has been designed first, to eliminate loss of water from a broken hydrant and the necessity of shutting off the supply over a portion of the distribution system while the hydrant is being replaced, and second, to reduce the cost and time required for repairs. These hydrants can be quickly repaired with cheap parts which can be kept in stock with little investment.

The tilting disc check valve is relatively new in the United States and possesses some advantages for use on pump discharge lines or other locations where reduction of water hammer is desirable.

Several new types of automatic cone type plug valves are on the market. Valves of this type provide adequate protection to pumps and piping, but their high cost in the past has retarded their general use.

Lubricated plug valves are now made by a number of valve manufacturers and, while they have been largely developed for the oil refining industry, may find applications in the water works field. They are of more value, however, for service pressures in excess of that ordinarily found in water works practice.

ELEVATED TANKS

One of the more noticeable and conspicuous changes in the water works systems throughout the country is apparent in the appearance of elevated water storage structures. The evolution of elevated storage has progressed through various stages of stand-pipes, (1) flat bottom wood tanks on steel towers, (2) hemispherical bottom steel tanks with a total tank depth several times its diameter, (3) ellipsoidal or segmental bottom tanks with a total depth not greater than the tank diameter to a design (4) which is, in effect, flat bottom construction supported by beams or columns and which is more or less independent of the tank diameter, providing a type of construction with no structural limit to capacity and with a tank depth entirely within a reasonable and useful depth range. Thus it is now possible to obtain

a two or three million gallon capacity elevated tank which has a depth of twenty-five feet or less, whereas fifteen or twenty years ago a one million gallon capacity tank probably would have had less than one-half its capacity in the upper twenty-five feet of the tank. This means that if the entire capacity were available at the desired pressure, the modern tank with a depth of twenty-five feet could be built about twenty-five feet lower than the old type, with an equal saving in pumping head and a consequent reduction in pumping costs.

Great progress has also been made in improving the appearance of elevated steel tanks through careful design from an architectural and aesthetic standpoint. This is also true of masonry enclosed standpipes and tanks.

METERS

There have been no recent radical changes in the metering of water, although most types of meters show refinements in design to provide better durability and greater accuracy. The use of current meters is increasing, due probably to price considerations.

One of the fairly recent developments in hydraulic instruments is the time-impulse type of electrical transmission for meters, depth gauges and similar devices. The time-impulse principle is better suited to long distance transmission than other types commonly used for this service for the following reasons:

1. It requires not more than two wires.
2. Its accuracy is not affected by resistance or induced currents in the transmission line.
3. Being independent of phase synchronization, the transmitter and the receiver can be energized from different sources of electrical energy.
4. No practical limit to distance between transmitter and receiver.

CHEMICAL FEEDING DEVICES

Devices have been developed to meet the demand for the newer chemical treatments of water, most of them being adaptations of existing equipment to meet special conditions. Solution type feeders have been designed to feed solutions at a constant rate in order to get away from trouble caused by clogging of orifices and corrosion of metal used in regulation of flow. A simple type consists of a tank with a flexible discharge outlet which is lowered, by means of a

synchronous electric motor at a constant rate. The movable outlet, being open throughout its length, has little tendency to clog.

The amount of chemical used is varied by the solution strength. It is customary to install a chemical dissolving tank above the feeder, this dissolving tank being filled with one day's supply of chemical and just enough water to fill the feeder tank. Machines are also available to feed automatically any chemical solution in exact proportion to the flow of water through the plant.

Because weight is a more constant and dependable unit of measurement than volume, especially with materials like pebble quicklime, which may contain a variable amount of fine material and dust, dry feed equipment has been developed to proportion and feed dry chemicals at a constant rate, and control such rate by the amount of chemical being fed. Any momentary variation in the rate of feed from the feeding device is immediately compensated for and corrected. For this reason, it is possible to use a simple type of feeder, which in itself is not particularly accurate, but whose errors are being constantly corrected. This feature makes it possible to use a feed device which is not as subject to clogging as a more accurate volumetric feeder would probably be. Equipment of this type provides several features difficult or impossible to obtain accurately in feeders of the orifice, volumetric or displacement type. First, an accurate chart record of the actual rate of feed over any time interval can be obtained. Second, the total amount fed over any time interval can readily be obtained. Third, this information can be obtained where the feeders are supplied directly from large storage bins too large to support economically on scales and where any exact information relative to daily amounts of chemical is difficult to obtain, especially where the rate of feed changed several times a day.

AUTOMATIC ELECTRIC CONTROL

Much progress has been made in automatic electric control of water works equipment, ranging from accurate automatic control of booster pumps to complete automatic operation of an entire filter plant and pumping equipment. Complete automatic control of filter plant operation would probably not be desirable except for small plants operating only a few hours a day, but some automatic control would seem to be desirable in any plant. Some small towns do not have enough money to hire a full time operator, and too often

one man is expected to look after the entire system, including meter reading and taking care of house services. In such cases, the time the operator can spend at the filter plant and pumping station is very limited. In a case of this kind, it may be desirable to provide automatic operation of the entire plant, or possibly automatic stop only. The latter is more desirable, as the operator can check the conditions at the plant before putting it into operation and then leave it to the automatic control to stop all operations when the elevated storage becomes full. It is necessary to keep the chemical feed equipment supplied with chemicals and keep the filters washed, so even in case of full automatic operation, it is necessary for the operator to spend some time at the plant each day. There would be practically no difference in automatic operation and that requiring manual starting with automatic stop except that in the case of the latter, an ordinary three-wire start-stop push button station is installed in the pressure switch circuit in such a manner that when the equipment is stopped with the pressure switch, it cannot start until the start button is manually closed, even though the pressure switch is in a closed position.

The plant control, under these conditions would be along the following lines:

1. The water supply to the plant, if by gravity, would be controlled by means of a throttling type float valve regulated by the water level in the basins or filters. If the supply is furnished by means of pumps, they would be controlled by means of the same equipment as is used for controlling the operation of the high service pumps.

2. The flow of water through the basins and filters would be controlled by the water level in the filtered water reservoir. Automatic high water level shut-off control on the filter effluent rate controllers would shut down the filters when the reservoir became full. Closing the rate controllers would stop the flow of water through the plant.

3. The high service pumps would be controlled by the water level in the elevated storage, either with full automatic operation if it was desirable to maintain the storage level within a close range for fire protection, or with automatic stop and manual start if the elevated storage was sufficient to permit this kind of operation. In order to prevent emptying the filtered water reservoir in case the pumps gained on the filters, or if a failure in the supply occurred, a float

switch operating on the clear well water level should be connected into the pump control circuit in such a manner that the pumps would be stopped before the suction supply was exhausted.

4. All chemical feed equipment would be electrically operated or controlled through a magnetic switch which would be controlled by the same pressure switch that controls the pumps. The pressure switch can be connected to control the operation of a multi-pole magnetic contactor, one circuit of which would control the pilot circuit of each pump motor starter and one circuit which would be in the supply circuit serving the chemical feed machines and one or more normally closed solenoid valves in the water supply line or lines serving the chemical feed equipment.

Automatic filter washing is possible with equipment now available. Robot control for filter valves can be obtained for either hydraulic or electric motor operated valves. Practically any desired valve sequence or time-function combination can be obtained and it is possible to start the entire sequence in operation by performing one manual operation or this can be done automatically through the loss of head in the filter. This last would provide complete automatic washing of the filters at one predetermined loss of head.

The application of automatic control to any process is one which should receive attention by the designer, as in many cases, automatic control is more reliable and accurate than manual operation.

FILTER MATERIALS

Due to the gradual depletion of satisfactory gravel from deposits worked by commercial gravel plants, as evidenced by the increasing difficulty of such plants in meeting gravel specifications and the lowering of quality of the gravel furnished, engineers should be interested in the development of filter bottoms which will reduce or entirely eliminate the filter bed gravel. One difficulty in the entire elimination of gravel is to get a porous or perforated medium which will not clog or become filled with deposits precipitated from the water. It is not difficult to obtain sands with well rounded grains up to one-fourth inch in grain size from our river sand plants, and filter bottoms which would retain material of this size should be satisfactory with most waters.

We have for a number of years successfully used river run sand taken from the Kaw River, with no other preparation than the removal of the finest material down to a maximum limit of two to

twelve per cent passing a No. 40 sieve. We have been specifying that all of this material pass a No. 10 sieve with the idea of passing from the coarsest sand gradation to the finest gravel gradation, which is to be retained on a No. 10 sieve, without much of a break in size. This gradation toward the coarse end of the size gradation could readily be extended to one-fourth inch or coarser without difficulty. Because the sand soon becomes segregated and classified according to size and specific gravity after a few washes, the coarse material would automatically stratify on the porous filter bottom. It is our opinion that since the sand lying in any horizontal plane in a well washed filter bed has a uniformity coefficient approaching unity, that a low uniformity coefficient throughout the entire sand bed is neither necessary or altogether desirable, due to the abrupt break between a sand of low uniformity coefficient with the top gravel layer, and that a uniformly graded sand will give good results provided the proportions passing a No. 30 sieve are carefully controlled to give a proper filtering layer on top of the filter.

CONCLUSION

In conclusion, I wish to point out that I may, in some instances, have mentioned materials or devices which are merely new, and because of a lack of time or through limited use, have not as yet proven themselves to be actual improvements. The lack of proven performance over a long period of time should not necessarily mitigate against their use, however, if their basic principles of design are correct, because if nothing were to be used until it could show successful performance over a long period of service, progress would be slow indeed.

NEW WATER SUPPLY FOR LITTLE ROCK, ARKANSAS*

By Chester A. Smith

(Consulting Engineer, Burns & McDonnell Engineering Company, Kansas City, Mo.)

PRESENT SUPPLY CONDITIONS

The Arkansas River has been the main source of water for almost sixty years and at present is serving North Little Rock and Little Rock, a combined population of over 105,000. The River rises in the Rocky Mountains of Colorado. However, before this clear soft mountain water arrives at the water works intake at Little Rock, it has received additional waters from its many tributaries. The water has accumulated a heavy turbidity content from the recently famous "dust bowl" area of eastern Colorado and western Kansas, carbonates and sulphates from hard water areas of Kansas, Oklahoma and Arkansas, various forms of pollution from cities and populated lands on its drainage areas, and, also in the past 30 or more years, a serious added contamination from the salt water and oil wastes of the Mid-Continent oil fields.

For the year 1935, the average turbidity of the river water was 980 p.p.m. with a maximum of 15,200 p.p.m. Turbidities exceeding 50,000 p.p.m. have been noted in previous years.

In 1934, a drought year, the chloride content averaged 377 p.p.m. with a maximum of 1250 p.p.m. There were 139 days in 1934 in which the chloride content exceeded 400 p.p.m., the limit adopted by the company for potability of the domestic supply. In order to reduce chlorides to 400 p.p.m., an auxiliary well supply, located in the river valley near the pumping station, is mixed with the river water prior to treatment and purification. The water from individual wells contains from 300 to 400 p.p.m. of carbonate hardness and from 6 to 19 p.p.m. of iron. Thus when the well supply is used, lime is added to reduce hardness and iron.

*Presented before the Missouri Valley Section, October, 1936.

The present sources have been treated to render a safe water from a hygienic standard. However, the slight saline tastes, odors and tastes from organic sources and the hardness content have been the cause for complaints for years and have kept alive an agitation for a better water supply.

Mayor R. E. Overman was elected in the fall of 1934 on a platform pledging the securing of a new water supply and municipal ownership of the waterworks system.

PRELIMINARY STUDIES AND ESTIMATES ON NEW SUPPLY

The Chamber of Commerce Water Committee, with some funds to defray incidental costs, and with W.P.A. labor making the preliminary surveys under the direction of Dr. George C. Branner, State Geologist and M. Z. Bair, State Sanitary Engineer, had prepared a preliminary report and an estimate upon a gravity supply from Alum Fork of Saline River.

Even before taking office in the spring of 1935, Mayor Overman began preliminary negotiations for the purchase of the water system and investigation of ways and means for financing the cost of the new supply.

Negotiations for purchase failed temporarily but an agreement was made with the company which provided that the city would finance and build the new supply and the company would purchase a certain annual amount of water at a sum sufficient to pay interest and retirement charges on the investment plus normal maintenance costs, and all over the fixed amount of water at a fixed rate for each thousand gallons.

Burns & McDonnell Engineering Company were then retained in August to study the project, review the preliminary report and estimates and to gather such additional data necessary to prepare a report, plans and estimates for filing an application for a P.W.A. loan and grant under the 1935 P.W.A. act. With much too short a time limit, working with temperatures hovering around the century mark, tramping over a 35-mile pipe line through rough, rocky, densely wooded terrain, amply infested with wood ticks, chiggers, tarantulas, with occasional rattlers and moccasins to warn one to keep moving, the application was filed on September 4th, 1935 with the State P.W.A. office—just within the time limit for filing P.W.A. applications.

Notice of approval was received by the city and work started on

final surveys on October 1st, 1935. About six weeks later, the city was notified that approval was rescinded, so all work was stopped. Mayor Overman immediately started negotiations for financing the project by sale of revenue bonds when notified by Washington authorities that the project was reinstated under the 1934 Act with 30 percent grant and 70 percent loan. The city accepted the grant but sold the revenue bonds to private parties. The city and company reached an agreement on $3,850,000 for the value of the water works property in Little Rock. Since April 1st, 1936 the city has been operating the system and the new supply is under construction.

NEW SUPPLY

The new supply will consist of an impounded lake located on Alum Fork of the Saline River approximately 32 miles due west of the present filter plant. The names, Alum Fork and Saline River, are not especially suggestive of a good water supply source. However, analyses of samples from the natural flow of Alum Fork show no appreciable mineral content, especially chlorides or aluminates. The average hardness of the water is only about 10 p.p.m.

The lake will be filled with the runoff of the drainage area and to use the words of a local newspaper description, the new project will "hitch the water faucets to the rain clouds."

The new soft water supply will produce an estimated annual saving of approximately $200,000 in the use of soap, plus additional savings of thousands of dollars in plumbing repairs, fuel and other items of expense chargeable to hard water.

The lake will be approximately four miles in length, measured along the present stream course, and will have a surface area of two square miles, with a shore line of over seventeen miles. The maximum water depth will be one hundred feet. The estimated usable storage is over fourteen billion gallons, which with the estimated minimum annual runoff in three successive extreme drought years, will be sufficient to supply the estimated future water requirements of 17,500,000 gallons average daily use for both North Little Rock and Little Rock.

The drainage area of the water shed including the lake area when filled is approximately 42.5 square miles, located in the Ouachita National Forest. The average annual rainfall is 48 inches. The normal runoff, considering the character of the water shed, allowing for evaporation, transpiration and other losses, is estimated as

seventeen and one-half inches, producing a yield of 14 billion gallons. A possible minimum rainfall of 25 inches which is lower than any past record, will produce a runoff of 5.35 inches or 4 billion gallons.

The new supply will be brought to the present purification plant by gravity through approximately 32.4 miles of 39-inch pipe supply line, having an estimated daily capacity of approximately 25,000,000 gallons, or the estimated daily consumption during a maximum month in 1975.

DAM

A rolled earthen dam proved to be the most economical type for construction after comparisons of preliminary designs and estimates on various forms and types of masonry dams.

The dam under construction is approximately 2800 feet long including the "around the end" spillway section and dikes. The embankment slope on water side varies from 2 to 2.75 horizontal to 1 vertical, and on the down stream side from 1.8 to 2.5 horizontal to 1 vertical with 10 foot berms at every 35 feet vertical distance from the crest. The crest width is 20 feet with a maximum bottom width of approximately 550 feet at the maximum height of 115 feet above bed rock. The center section with slopes 1 to 1 will be constructed of impervious material, with the down stream section of pervious material and the upstream section of pervious or semi-pervious material. The total earth yardage as compacted in the dam is estimated at 1,530,000 cubic yards. The water face of the dam will be protected with rip rap masonry and the down stream slope will be sodded or sprigged with Bermuda grass, and will be provided with gutters and drains for removing runoff drainage to prevent excessive erosion.

The geological formations at the site of the dam consist of strata of shale and sand stone formations tilted at over 45 degrees with the horizontal and dipping in the upstream direction. Drillings into this formation indicated closely packed solid strata to a considerable depth with no intervening stratum of clay or faults filled with porous or pervious material. Dr. Branner, State Geologist, approved the site from a geological standpoint as a safe foundation upon which to construct an earth filled dam. The rock and shale formation, except in the present stream bed, is covered with from 1 foot to 18 feet of deposit consisting of clay, sand and gravel in varying depths and mixtures. This overburden will be removed beneath the impervious

and upstream pervious or semi-pervious sections and the compacted fill placed directly upon the bed rock formation. A concrete diversion conduit of horseshoe shape, approximately 10 feet diameter will be constructed through the west end of the dam for diversion of average normal floods during construction of the dam and upon completion of the dam this will be plugged with concrete at the upper end. After the reservoir is filled, this conduit will provide access to a system of pipes that will extend upward into the body of the dam whereby the line and slope of saturation may be observed and determined.

The specifications provide that the earth embankment shall be placed in layers of not exceeding 8 inches in thickness, shall be of a proper moisture content and shall be rolled with at least 8 trips with a "sheeps foot" roller weighing not less than 1100 pounds per linear foot of tread. Compaction will be tested with a plasticity needle in conformance with the "Proctor Methods" (See article by R. R. Proctor, Vol. 28, No. 1, Journal A. W. W. A.). The various depths and compaction shall require a needle reading of from 400 to 2300 pounds per square inch and a dry weight of compacted material from 110 to 124 pounds per cubic foot.

One of the problems involved was to locate sufficient material of the proper character to place in the impervious section of the dam. Nine hundred and eighty-nine test pits were dug in locating the proper borrow areas. Laboratory tests were made to determine the stability and imperviousness under varying degrees of compaction and the proper moisture content under which each class of soil would be placed in embankment. Twenty-five different soils were so tested. The test results indicated that soil existing in borrow pits weighing 90 to 95 pounds per cubic foot dry weight will be placed and compacted in the dam at weights between 110 to 120 pounds per cubic foot. Also the tests indicated that the deepest soil existing at the dam site would settle about 2 feet if the weight of the dam were loaded upon it, hence all soil will be removed to bed rock and placed under the specified rolling and compaction tests.

SPILLWAY

The topography at the east end of the dam was well adapted for an "around the end" spillway. A natural saddle existed between two small tributaries to Alum Fork, one above and one below the dam. The weir section of the spillway is 300 feet wide at an elevation 15 feet

below the dam crest. The approach is fan shaped tapering toward the weir. The wasteway decreases in section away from the weir as the velocities increase due to the increasing gradient as required by topographical conditions. The discharge into the spillway canyon or ravine is at a point where the topography forms a natural stilling basin. The spillway canyon discharges into Alum Fork over 500 feet below the down stream toe of the dam with a natural rock bluff deflecting any current tending to erode toward the dam site.

The spillway excavation is carried into solid shale and rock formation and the spillway will be lined and paved with concrete.

Careful studies were made as to the required capacity for the spillway. The greatest possible runoff or flood conditions were assumed and careful estimates were made as to storage effects of the lake during such heavy runoff periods.

A spillway capacity of 30,000 second feet was adopted. The hydraulics of the spillway when flowing at various rates of discharge were carefully figured, checked and re-checked. After the spillway had been tentatively designed, a model spillway at 1/100 scale was constructed in the hydraulic laboratory at Iowa State College, Ames, Iowa, including the approach conditions and topography of the stilling basin and canyon through which the water will return to Alum Fork. The model was tested at varying rates of discharge and all hydraulic features, as wave action, hydraulic jumps, velocities, depths, action in stilling basin and outlet canyon, were carefully noted. The curvature of the side walls below the weir as designed, required some minor correction which reduced the effect of some cross wave action, otherwise the model demonstrated the correctness of the computations and design. The model also demonstrated that the spillway would carry a discharge of 40,000 second feet without endangering the main dam structure.

SUPPLY LINE

A 39-inch internal diameter steel core reinforced concrete pipe line as manufactured by the Lock Joint Pipe Company is being constructed from the present filter plant to the lake, a distance of approximately 167,000 feet including two reinforced concrete lined tunnel sections of about 1,100 and 1,500 feet in length. There is a fall of 230 feet from full lake elevation to water elevation in the first settling basin at the plant. Although the line will be a gravity flow line, due to the mountainous character of the country through which the

line passes, the pressure heads on the line will vary from practically nothing to about 300 feet. The line is designed for an additional 50 foot head at the dam end in order that at some future time when additional capacity may be required, a booster pump may be used at the dam to increase the hydraulic gradient and thereby increase the capacity of the line by about 15 percent. The line is well provided with air relief valves and combination air relief and vacuum valves at high points, some 176 valves in all. There are 58 blow-off connections and 81 manholes for construction and inspection purposes.

In most cases where the pipe line crosses small streams or washes, the trench will be excavated into the solid shale or rock formation, usually exposed in the bed of the streams, and the pipe will be protected by a concrete encasement filling the rock trench and covering the pipe. Only in a few cases will the pipe be carried across ravines on short reinforced concrete bridge structures.

The upper end of the supply line connects with a control gate tower located in the lake toward the east end of the dam. The usual provision has been made for drawing water at various depths by the spacing of the sluice gates in the central tower. A venturi meter and a cone valve will be installed in the line near the dam outlet for measuring and controlling the average daily required amount of water to be delivered through the line to the purification plant and city system. A venturi meter will be installed in the line near the outlet to the settling basins. An overflow pressure break is provided between the meter and the cone valve in the lower end of the line and the valves controlling the flow into the settling basins to prevent any possibility of subjecting the 32 miles of pipe line to heads above the designed hydraulic gradient heads or against water hammer through closing the outlet valves. No other valves are to be installed between the upper control valve and the regulating valves below the pressure break. To have designed the line for the static head from the lake water level would have increased the cost of the pipe line by approximately $175,000 to $200,000.

MISCELLANEOUS ITEMS

In connection with the surveys, designs and plans for the main contracts on dam and pipe line construction, there were the securing of some 30 miles of rights-of-way, the purchase of lands that will be inundated by the proposed lake, the moving of a cemetery and re-

building a church, the design of operator's cottage with water supply, a small hydro-electric lighting plant, telephone line or radio communication between the city plant and dam, a fish rearing pond in lieu of a fish ladder, a bridge over the spillway channel, about 5.75 miles of U. S. Forest highway to be relocated and constructed and many other minor details.

Bids were received upon the pipe line contract on June 16th, 1936. Three types of pipe, namely, cast iron, steel and reinforced concrete were bid upon by 8 contracting firms. The award was made to the low bidder—Lock Joint Pipe Company, Ampere, New Jersey—for a total sum of $1,562,682.72. The pipe is being made in a plant located in the outlying district of Little Rock and the first pipe laid in trench was on October 2nd, 1936. The plant is now manufacturing 40 pipes, 16 feet in length per day and expect to increase production to at least 50 pipes per day.

Five bids on road relocation were received on August 27th, 1936 and contract was awarded to M. E. Gillioz, Monett, Missouri, low bidder, for $39,869.59.

Nine bids were received on the dam, spillway and appurtenances contract on August 28, 1936 and the award was made to L. O. Brayton & Company, Dyersburg, Tennessee, the low bidder, at $921,726.05.

Construction camps are being erected for the latter two projects and active construction work will commence by the early part of October.

It is contemplated that the contract for the clearing and necessary grubbing of the lake area will be ready for receiving bids early in 1937.

Possibly a small contract will be let for the telephone line unless radio communication is adopted for which experimental tests have already been made by the radio division of the Little Rock Police Department.

It is estimated that the total completed cost of the water supply project including lands, rights-of-way, construction work now awarded and to be awarded, interest on bonds during construction, legal and engineering costs, general administration costs will approximate $3,100,000. The project should be completed and the new supply available early in 1938.

Mayor R. E. Overman has been untiring in his efforts to fulfill his

platform promises for a municipal water system and a new pure soft water supply, though in the early stages the project was beset with many difficulties and delays.

Burns & McDonnell Engineering Company are the designing and supervising engineers. Mr. Louis C. Hill of Los Angeles was retained by the city at the request of Alexander Allaire, State Director of P.W.A., to review the project before final contracts were awarded.

WATER HAMMER: CORRECTING AN UNUSUAL PROBLEM*

By Vance C. Lischer

(Engineer in Charge of Production, St. Louis County Water Company, University City, St. Louis, Mo.)

The increasing use of electric power for driving centrifugal pumps has forced into the water works field recognition and consideration of the phenomenon of water hammer which previously had been a problem confined largely to the hydroelectric power field. The American Society of Mechanical Engineers, realizing the lack of a comprehensive resumé of both present and past studies on the subject in this country and abroad, have a Committee on Water Hammer which has published a very valuable symposium on the subject. Study and knowledge of water hammer in pump discharge lines admittedly lags what has been done in connection with penstock surges where valve closure takes place at the outlet end of the line instead of at the inlet end, as is the case in pump discharge lines.

In the operation of electric driven pumps instantaneous interruption of motivating power can occur—due to causes beyond control. Voltage fluctuation due to lightning is the most common cause for circuit breakers opening both at the source of power and at the pumping station. Rapid closure of the discharge valves in these cases is necessary to prevent reverse rotation of the pumps. Conditions such as these affect no other type of prime mover.

Unless means are provided to prevent damage to pipe lines the continuity of operation of such a water plant becomes an uncertainty and a source of worry to those responsible for its operation. The operation of the main pumping station of the St. Louis County Water Company, prior to the correction of its water hammer problem, was an excellent example.

PLANT DATA

The main pumping station is located at the purification plant on the Missouri River and pumps the finished water a distance of

* Presented before the Missouri Valley Section, October, 1936.

PLAN of PIPE LINES
PROFILE of TRANSMISSION MAINS — St. Louis County Water Co.
Elevations Based on City Datum
Distances from Plant Reservoir in Feet
Reservoir
Main Pumping Station
30" Cast Iron Pipe
24" Cast Iron Pipe
36" High Test Cast Iron Pipe
Class B
Class C
Class D
Hydraulic Gradient
Barometric Limit of Pressure at High Point

FIG. 1

39,100 feet to the highest point in the distribution system where a reservoir and booster station are located. (See Plan and Profile, Fig. 1.) The difference in elevation between the clear water reservoir on the Missouri river and the reservoir on the high ground is 266 feet. The transmission mains consist of parallel cast iron pipe lines 20 inches, 24 inches, and 36 inches in diameter, bypassed at frequent intervals along the route. The pumping equipment consists of two identical electrical driven centrifugal pumping units of three stages each with a combined capacity of 36 m.g.d. The first stage of each unit is driven by a 550 h.p. wound rotor variable speed induction motor, and the second and third stages are driven by a single 1100 h.p. synchronous motor. The check valves employed on these pumping units are 24-inch x 16-inch Larner-Johnson, needle-type. Motor driven gate valves on the discharge of each pump are also provided.

At the time the pumps were installed in 1931 there were four 8-inch spring loaded relief valves installed at the Pumping Station for the purpose of limiting the pressure rise due to water hammer. They were set to open at a pressure slightly higher than the maximum operating pressure. These valves were complete failures. Valves of this type cannot function fast enough to accomplish any good for this purpose.

PIPE LINE FAILURES

The tremendous forces that exist when surge or water hammer takes place in pump discharge lines were evidenced in pipe line failures which occurred quite frequently at this plant before the condition was corrected. To those in charge the failures caused justifiable anxiety concerning the maintenance of continuous service to consumers. Reserve supply and standby equipment had to be placed in service during the repair of many of these failures. Some of these happenings are here recounted.

At 12:30 A.M. one night a large piece was blown out of a perfectly sound 6-inch 90° ell (see Fig. 2). This ell is on a fire plug lead off of the 36-inch pipe line on the high point approximately 3,000 feet from the pumps. The 36-inch line had to be taken out of service for 14 hours to replace this bend.

At 2:35 A.M. on another night the bonnet of the shut-off valve on the lead to this same fire hydrant was blown off. The bonnet was perfectly sound and the break was in the cast iron. In both

FIG. 2. BROKEN 6″—90° ELL

FIG. 3. REPAIRING 20″ PIPE LINE FAILURE
Broken pipe is shown in foreground

these cases the broken parts were blown completely out of the ground.

On three occasions a 12-inch branch line in the vicinity of this fire hydrant burst. All of these breaks were violent, full lengths of pipe being split in most instances. On one of these occasions the bourdon tube of a recording pressure gauge reading up to 230 pounds was damaged beyond repair due to the excessive pressure. The static pressure where this gauge was set is 35 pounds.

A newly set fire hydrant on this same branch line was pushed off the end of its lead three times before clamps strong enough to hold it were installed. After it was securely fastened the barrel of the hydrant was split from end to end due to a subsequent surge. Most of these occurrences happened during the night.

At 1:15 A.M. on a fourth of July the 20-inch transmission line burst at a point about 1,000 feet from the pumps, part way up the river bluff. (See Fig. 3.) A section of pipe was demolished. At the time this happened the bourdon tube of a recording pressure gauge which was installed on the high point 3,000 feet from the pumps was completely damaged by the excessive pressure which caused this break. The gauge read up to 185 pounds; the static pressure at that point is about 35 pounds.

At 10:13 A.M. one day a 36-inch x 20-inch reducer about 100 feet from the pumping station was pushed 6 inches out of home. (See Figs. 4, 5, 6.) It took approximately one hour to shut down the two 36-inch valves necessary to isolate this leak. The shut down also isolated the electric pumping station and it was necessary to operate the standby steam plant for 10 hours until the leak was repaired. Figure 6 shows the reducer out of home after the water had been pumped down.

Besides these major failures there were many joint leaks not of a serious nature in themselves, but which were an indication of movement doubtless due to the repeated high pressures.

The expense caused by this weakness of the system is shown by the account, Maintenance of Transmission Mains, before and after correction of the difficulty. Before correction of the problem, from January 1932 to June 1934, this expense amounted to $3,769; and afterwards, from June 1934 through August 1936, $1,471. None of this expense since the correction has been caused by failures due to water hammer.

FIG. 5. VIEW OF 36″ x 20″ REDUCER LEAK BEFORE WATER WAS SHUT OFF

FIG. 4. CLOSING OFF SUPPLY TO 36″ x 20″ REDUCER

FIG. 6. VIEW OF 36″ x 20″ REDUCER LEAK SHOWING REDUCER OUT OF HOME

STUDY OF PROBLEM

The tests conducted and corrective measures recommended in this problem were the work of Mr. S. Logan Kerr, Chairman of the A.S.M.E. Committee on Water Hammer, in his then capacity as Water Works Engineer for the Baldwin-Southwark Corporation.

The preliminary investigations to determine the remedial measures necessitated an analysis using charts taken with a high speed pressure gauge. Figure 7 is one of these charts. Each division marked one hour is equal to eight seconds on this chart. Figure 8 is a replotted chart of several of these curves on rectangular ordinates. Figure 8 shows a distinct wave phenomenon with a definite period as measured by the interval of time between the successive crossings of the pressure wave and the static pressure line, the only exception being the first depressed wave. The time required for the wave to pass the static line successively is 21½ seconds. It should be noted that the pressure is depressed below the static line approximately the same amount as the pressure rise above static in successive waves, and that it would be the same if it were not for the damping effect which is obviously taking place. The pipe line disturbance exists for approximately 25 minutes. (See Fig. 9.) Figure 10 shows other replotted high speed pressure charts taken when different rates of flow were interrupted. The period of the wave can again be seen to be about 21½ seconds. This time factor will always be the same regardless of the rate of flow interrupted or the rate of closure of the valve. It is a characteristic of the pipe line and is a function of the elastic properties of the material of the pipe line and the water contained in it. It is the time required for the wave to travel from the valve at the pumping station to the reservoir 39,100 feet away and return. It is often referred to as the critical time. The wave leaves as a subnormal pressure and it returns as an abnormal pressure. The subnormal wave travels undiminished from the valve to some point between the pumping station and the reservoir, depending on the rate of closure of the valve, where its intensity falls off in a straight line until it is zero at the reservoir; the return high pressure wave starts at zero at the reservoir, increases in intensity in a straight line to the maximum at the same point, thence travels undiminished to the valve where closure was made. (See Fig. 1.)

It can readily be seen from Figures 8 and 10 that for similar rates of closure of the discharge valve the pressure rise is higher when a

higher rate of flow is interrupted. The most important fact in this study which can be seen from Figure 10 and better on the original chart, Figure 7, is an irregularity in the contour of the curve indicating a superimposed wave. This superimposed wave results in these pipe lines whenever the pressure drop is greater than 50

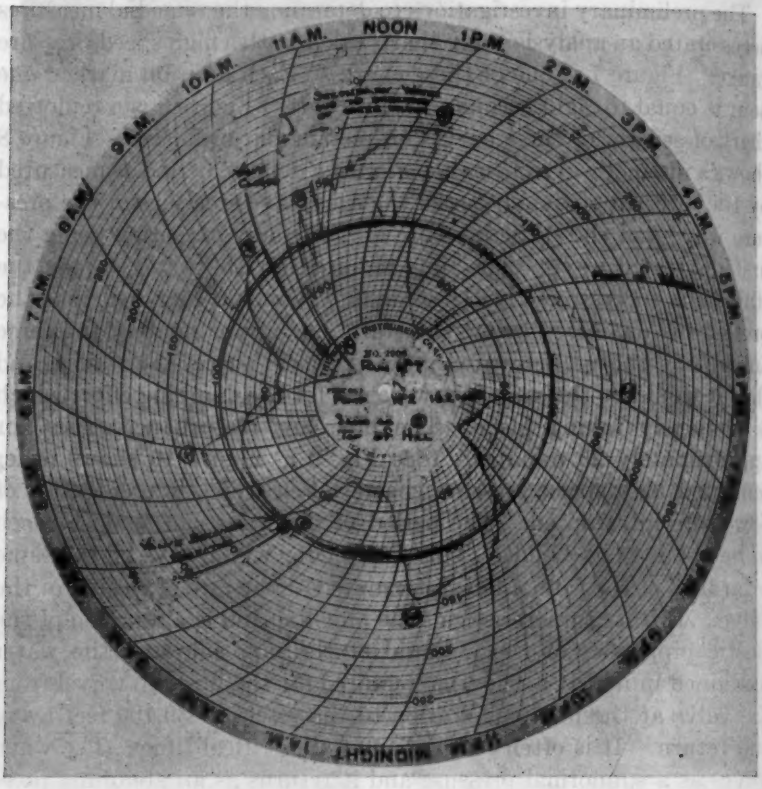


FIG. 7. ORIGINAL HIGH SPEED PRESSURE CHART

Gauge set at high point 3300′ from pumps. (Note superimposed wave.) No. 2 pump. Gate valve closed in 103 seconds. Pumpage rate: 16.2 m.g.d. Each division marked one hour is equal to eight seconds on this chart.

pounds. Under conditions when the drop is 50 pounds or greater, a physically impossible situation results at the high point 3,300 feet from the pumps. Subtracting the 50-pound pressure drop from the 35-pound static pressure at this point, the net pressure becomes absolute zero. Obviously for pressure drops near or greater than 50 pounds the water column at this point must part. As the pres-

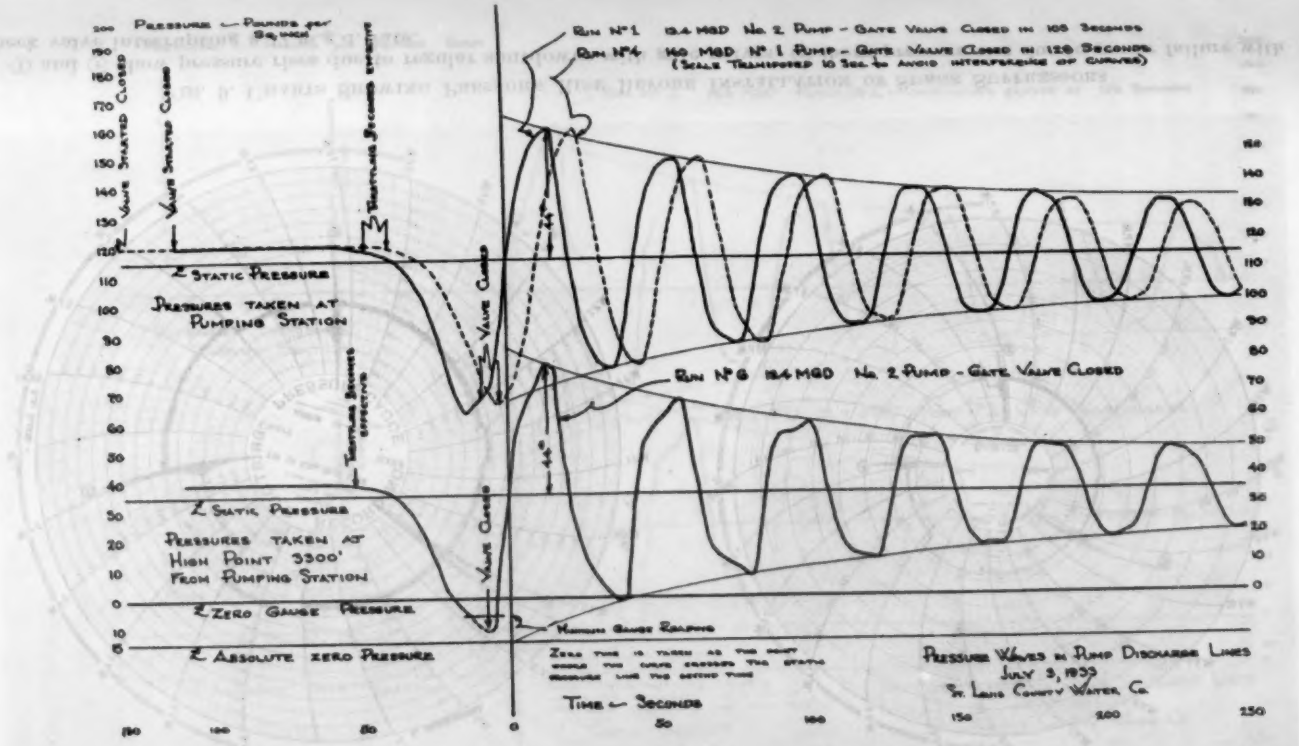

Pressure — Pounds per Sq. Inch
Run N°1 13.4 MGD No. 2 Pump - Gate Valve Closed in 105 Seconds
Run N°4 14.0 MGD N° 1 Pump - Gate Valve Closed in 120 Seconds
(Scale Transposed 10 Sec. to Avoid Interference of Curves)
Valve Started Closed
Valve Started Closed
Valve Closed
Static Pressure
Pressures Taken at Pumping Station
Run N°6 13.4 MGD No. 2 Pump - Gate Valve Closed
Static Pressure
Pressures Taken at High Point 3300' From Pumping Station
Zero Gauge Pressure
Absolute Zero Pressure
Time — Seconds
Pressure Waves in Pump Discharge Lines
July 3, 1939
St. Louis County Water Co.

FIG. 8

Chart at Pumping Station

Chart at High Point 3300′ from Pumps

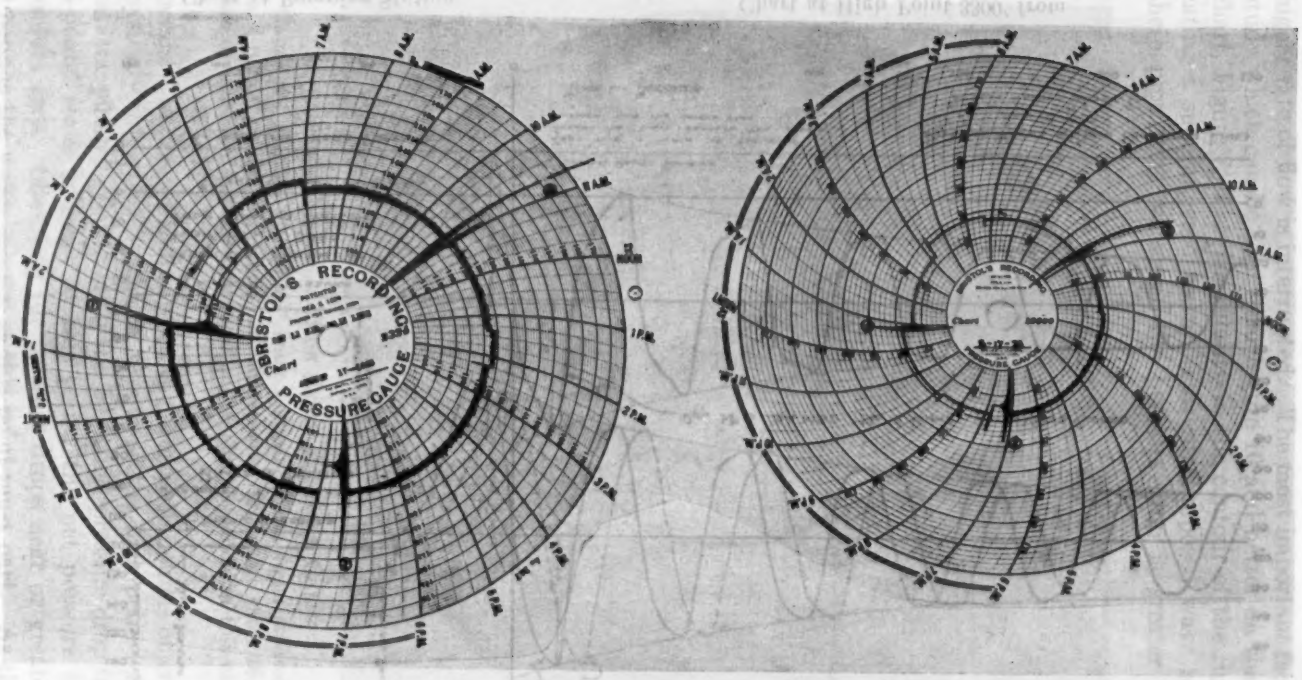


FIG. 9. CHARTS SHOWING PRESSURE RISE BEFORE INSTALLATION OF SURGE SUPPRESSORS

① and ② show pressure rises due to regular shutdowns with gate valves, ③ shows pressure rise due to power failure with check valve interrupting a 17 m.g.d. rate.

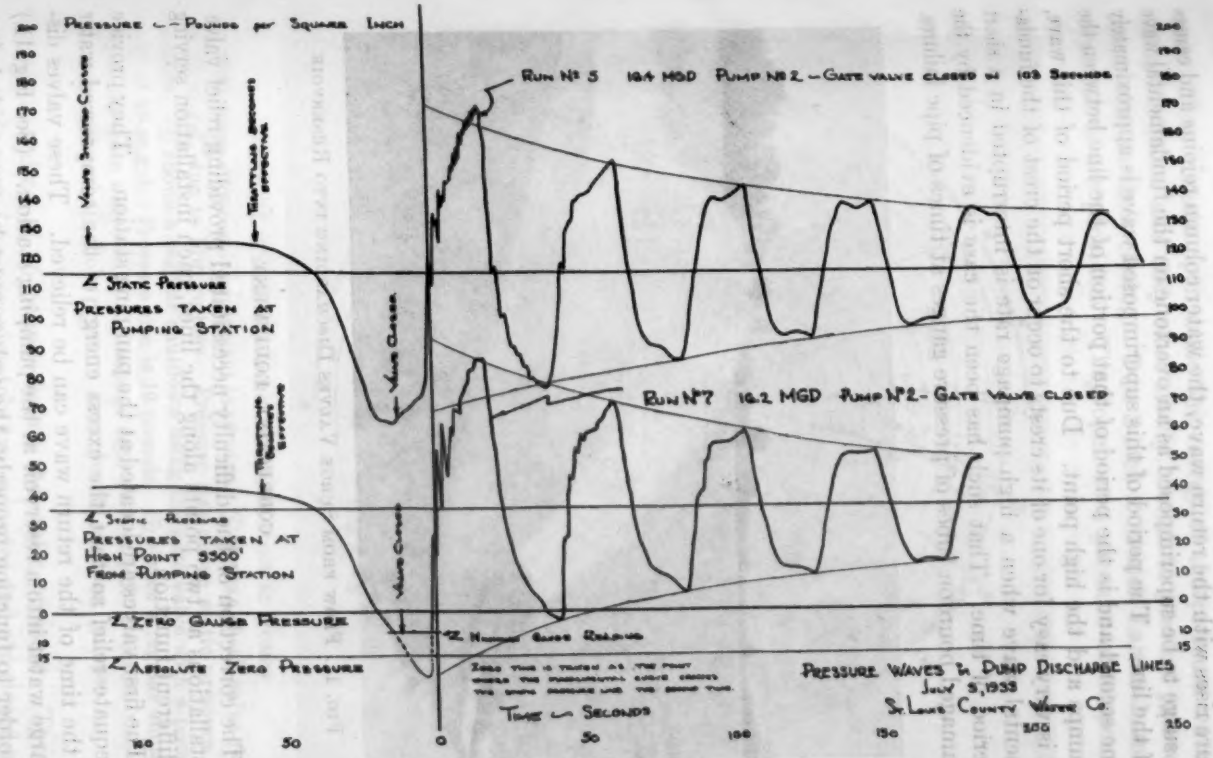

Pressure — Pounds per Square Inch
Run No 5 10.4 MGD Pump No 2 — Gate valve closed in 103 Seconds
Run No 7 10.2 MGD Pump No 2 - Gate valve closed
Valve Starting Closed
Valve Closed
Static Pressure
Pressures taken at Pumping Station
Pressures taken at High Point 5500' From Pumping Station
Zero Gauge Pressure
Absolute Zero Pressure
Time in Seconds
Pressure Waves in Pump Discharge Lines
July 5, 1933
St. Louis County Water Co.

FIG. 10

sure rises with the return wave, the water column rejoins and causes a surge to be superimposed as an overtone on the fundamental wave of the line. The period of this superimposed wave is approximately one second and is the period of that portion of the line between the pumps and the high point. Due to the short period of this wave, it is very easy for one of its crests to occur on the crest of the fundamental wave when a high pumpage rate is interrupted in a short period of time. That such has been the case is evidenced by the damaged bourdon tubes of pressure gauges at times of pipe failures.

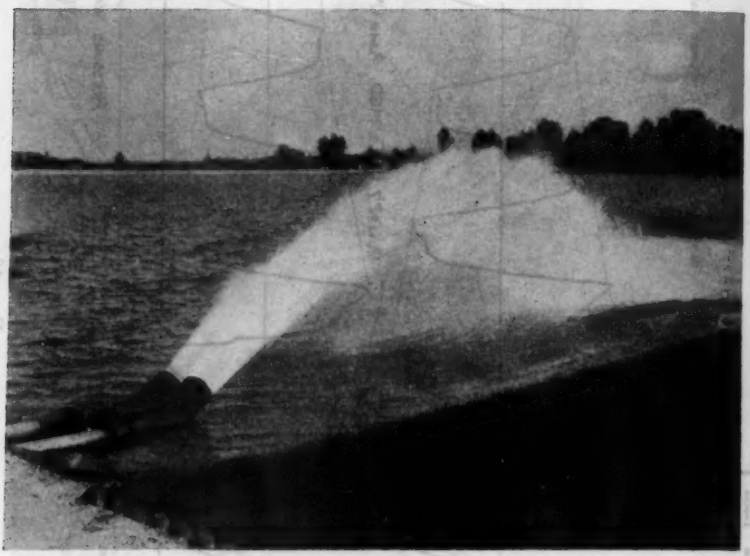


FIG. 11. FLOW FROM SURGE VALVES DISCHARGING INTO RESERVOIR

CORRECTIVE EQUIPMENT

The correction of the difficulty necessitated providing relief valve installations at two points along the lines, each installation serving a different function.

The first valves are located at the pumping station. They provide adequate relief so that the excess energy of the compressed water at the time of the return wave can be relieved. These valves discharge water into a reservoir at the pumping station. (See Fig. 11.) In order to function properly, the valves must be open wide before the return pressure wave reaches the pumping station. Therefore, they have controls which cause them to open with the drop in pres-

sure. They remain open while the energy of the pressure wave is being dissipated and they close slowly enough to prevent penstock surges due to their closing. Two 8-inch x 12-inch Larner-Johnson type valves are provided. They open at a pressure of 90 pounds

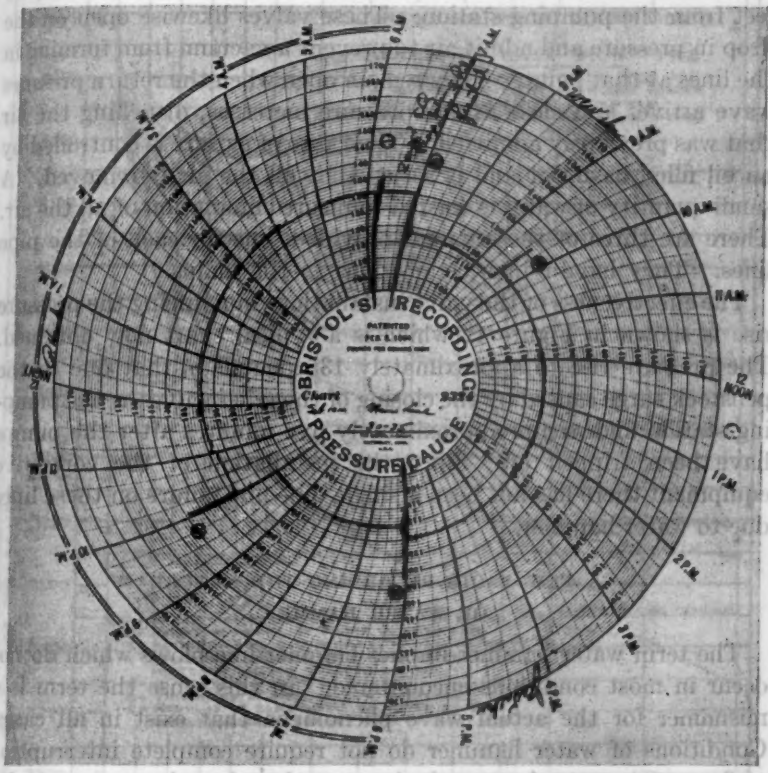


FIG. 12. CHART SHOWING PRESSURE RISE AT PUMPS

With surge suppressor equipment in service. ① shows a check valve closure at 19 m.g.d. due to low voltage. Pressure rise above static pressure is 17 pounds. ② shows a regular shutdown using the gate valve with a pumpage rate of 19 m.g.d. Pressure rise is 19 pounds. ③ shows a regular shutdown using the gate valve with a pumpage rate of 9 m.g.d. Pressure rise is 16 pounds. ④ and ⑤ show pressure rises due to starting pumps.

which is a drop of 25 pounds below the static pressure. They remain wide open for approximately two minutes and are closed again five minutes after the time they start to open. The maximum rate of flow through the valves as measured by a pitot tube at their dis-

charge into the reservoir is approximately 22 m.g.d. It is therefore obvious that slow closure of the valves is necessary to prevent penstock surges which could be as disastrous as those which are to be eliminated.

The second installation of relief valves is at the high point 3,300 feet from the pumping station. These valves likewise open on the drop in pressure and admit air to prevent a vacuum from forming in the lines at that point. They remain open when the return pressure wave arrives and close as the pressure increases, dispelling the air that was previously admitted. Their rate of closure is controlled by an oil filled dash pot and is set so that all the air is removed. A small quantity of water is wasted to insure elimination of all the air. There are three of these valves provided, one for each of the pipe lines. They have 6-inch connections to the mains.

The effectiveness of the valve installations in reducing the pressure rise is shown in Figure 12 which is a typical chart now obtained. The pressure rise to approximately 135 pounds is that due to the penstock surge caused by the closing of the relief valves at the pumping station and occurs approximately five minutes after the pumps have been tripped off. Since the installation of the corrective equipment there has not been a single pipe line failure on these lines due to water hammer.

A FEW FACTS REGARDING THE NATURE OF WATER HAMMER

The term water hammer implies hammer like blows which do not occur in most conditions encountered. In this sense the term is a misnomer for the actual wave phenomena that exist in all cases. Conditions of water hammer do not require complete interruption of flow. The wave is started whenever there is a change in velocity of flow. The phenomenon of water hammer in a simple system of a single conduit is fairly well understood. There is a remarkable correlation between theory and reality, especially in the determination of the velocity of the pressure wave. Figure 13 shows a chart prepared by K. W. Beattie (Jun. A.S.C.E.) published in the Supplement to the A.S.M.E. Symposium on Water Hammer, for the velocities of the pressure wave in various sizes and classifications of cast iron pipe. The velocity of the wave in the discharge piping system of the St. Louis County Water Company is 3,630 feet per second, which can be seen to correspond fairly well with the values shown in

the chart. The chart also gives the general formula used in its computation. This formula is theoretical and not empirical. It

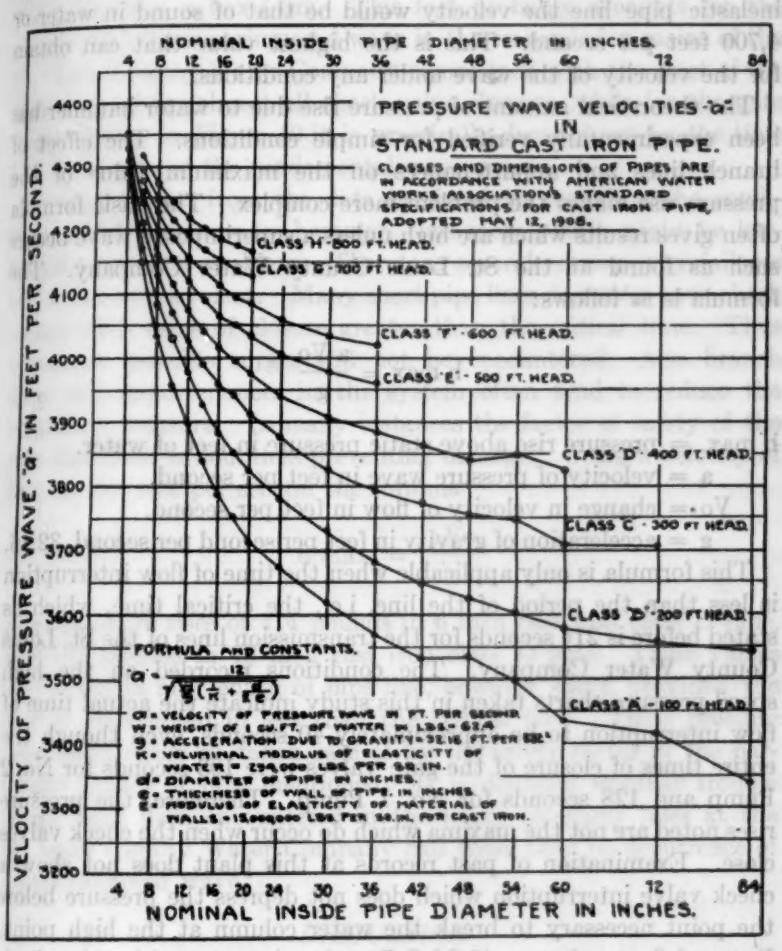

NOMINAL INSIDE PIPE DIAMETER IN INCHES.
4 8 12 16 20 24 30 36 42 48 54 60 72 84
PRESSURE WAVE VELOCITIES "a" IN STANDARD CAST IRON PIPE.
CLASSES AND DIMENSIONS OF PIPES ARE IN ACCORDANCE WITH AMERICAN WATER WORKS ASSOCIATION'S STANDARD SPECIFICATIONS FOR CAST IRON PIPE, ADOPTED MAY 12, 1908.
CLASS 'H' - 800 FT. HEAD.
CLASS 'G' - 700 FT. HEAD.
CLASS 'F' - 600 FT. HEAD.
CLASS 'E' - 500 FT. HEAD.
CLASS 'D' - 400 FT. HEAD.
CLASS 'C' - 300 FT. HEAD.
CLASS 'B' - 200 FT. HEAD.
CLASS 'A' - 100 FT. HEAD.
FORMULA AND CONSTANTS.
a = velocity of pressure wave in ft. per second
W = weight of 1 cu. ft. of water in lbs. = 62.4
g = acceleration due to gravity = 32.16 ft. per sec²
K = voluminal modulus of elasticity of water = 294,000 lbs. per sq. in.
d = diameter of pipe in inches.
e = thickness of wall of pipe, in inches.
E = modulus of elasticity of material of walls = 15,000,000 lbs. per sq. in. for cast iron.
VELOCITY OF PRESSURE WAVE "a" IN FEET PER SECOND.
4400 4300 4200 4100 4000 3900 3800 3700 3600 3500 3400 3300 3200
NOMINAL INSIDE PIPE DIAMETER IN INCHES.

FIG. 13. CHART FOR DETERMINATION OF WATER-HAMMER WAVE VELOCITIES IN CAST-IRON PIPE

By K. W. Beattie (Jun. A.S.C.E.), Research Assistant, I. P. Morris Division, Baldwin-Southwark Corporation, Philadelphia, Pa.

has been substantiated in test. The formula is identical with that for the determination of the velocity of sound in the same medium, namely, the pipe line full of water. The two phenomena are com-

pression waves which depend on the elastic properties of the substances in which they are transmitted for their velocity. For an inelastic pipe line the velocity would be that of sound in water or 4,700 feet per second. This is the highest value that can obtain for the velocity of the wave under any conditions.

The theoretical amount of pressure rise due to water hammer has been experimentally verified for simple conditions. The effect of branch lines and appurtenances on the maximum value of the pressure rise makes the problem more complex. The basic formula often gives results which are high unless a superimposed wave occurs such as found at the St. Louis County Water Company. The formula is as follows:

$$h\ max = \frac{a\ Vo}{g}$$

h max. = pressure rise above static pressure in feet of water.
a = velocity of pressure wave in feet per second.
Vo = change in velocity of flow in feet per second.
g = acceleration of gravity in feet per second per second, 32.16.

This formula is only applicable when the time of flow interruption is less than the period of the line, i.e., the critical time, which as stated before is 21½ seconds for the transmission lines of the St. Louis County Water Company. The conditions recorded on the high speed pressure charts taken in this study indicate the actual time of flow interruption to be approximately 40 seconds, even though the entire times of closure of the gate valves were 103 seconds for No. 2 Pump and 128 seconds for No. 1 Pump. Therefore, the pressure rises noted are not the maxima which do occur when the check valves close. Examination of past records at this plant does not show a check valve interruption which does not depress the pressure below the point necessary to break the water column at the high point. Rates of flow as low as 13 M.G.D., when interrupted by the check valves, have caused the water column to break. It is not possible then to verify the formula for maximum pressure rise on the basis of experience at this plant.

It should be noted in the formula for maximum pressure rise that the length of the pipe line is not a factor. This holds true only for flow interruption in less than the critical time. Also, the maximum rise is independent of the time in which the flow is cut off up to the critical time. The rate of flow interruption, when less than

the critical time, affects only the shape of the curve and not its maximum and minimum values. For instantaneous interruptions the curve assumes a box form. This fact probably accounts for the reason that the superimposed wave in the transmission mains of the St. Louis County Water Company compounds with the crest of the fundamental for almost all check valve closures, while for the gate valve closure in Figure 10 it is practically damped out by the time the maximum of the fundamental wave occurs.

It would seem from the data herein given regarding the conditions which cause water hammer, that more plants might be harrassed with that problem. There are several reasons why failures are not more prevalent. Many short pipe lines doubtless have check valves with rates of closure greater than the critical time. Thus maximum possible surges will not be encountered. Also branch lines and appurtenances in the system often tend to reduce the maximum pressure. In many instances the factor of safety of the pipe line itself is doubtless preventing damage. Using a velocity of flow of two feet per second the formula

$$h \text{ max.} = \frac{aVo}{g}$$

gives pressure rises of 114 pounds in 6 inch cast iron class D pipe, and 103 pounds in 48 inch cast iron class D pipe. These values may occur in any length of pipe line, however short, provided the flow is interrupted in less than the critical time. These pressure rises could easily be taken care of by the safety factor in the particular grades of pipe mentioned. It is doubtful whether trouble would have been encountered in the pump discharge lines at the St. Louis County Water Company had there been no occurrence of a superimposed wave.

DISCUSSION

S. LOGAN KERR (*Chairman, Committee on Water Hammer, A. S. M. E.*): The problem existing in the pumping mains of the St. Louis County Water Company was one of the most interesting encountered in the writer's experience. The unusual length of line, the use of three parallel mains with cross-connections, and the unfavorable profile all contributed to the critical condition which had been experienced. It was the writer's privilege to conduct the tests on the line, make the analysis of the difficulties and apply the correc-

tive equipment. The use of a high speed recording pressure gauge was invaluable in this investigation, as it demonstrated the true source of the difficulties and permitted a rational analysis to be made.

The problem would have been of simple proportions had the profile of the line followed a uniform slope from the pumping station to the reservoir. In this case a maximum surge would have been severe but would probably have been taken care of by the factor of safety in the pipe.

The pipe line rises rapidly from the pumping station to the top of the hill immediately adjacent, and then drops into a valley from which it again rises on a long gradual slope to the reservoir. This high point on the line immediately adjacent to the pumping station was the focal point of most of the breakages which occurred. The test data confirmed the preliminary analysis in locating a point at which the water column broke and then rejoined with an abrupt surge. The velocity assumed by the water immediately before the water column rejoined was extinguished instantaneously, and hence caused a surge consideably in excess of that which would have been experienced if the normal rate of flow in the conduit had been cut off.

It is possible to analyze existing installations or proposed projects for dangerous conditions and to provide corrective measures. It is often possible to apply simple protective measures which will limit the dangerous water hammer conditions which may be set up due to a combination of circumstances. The use of high speed recording pressure gauges, and the correct interpretation of the resulting charts can provide the basis for a sound analysis of pipe line problems.

It is important to realize in adjusting control valves that the total time of closure is not necessarily the limiting factor with respect to water hammer, but the rate at which flow is cut off does determine the maximum pressure rises which will be experienced. It is well known that the first half of the stroke of a gate valve from the open position has little or no effect upon the flow of water, but as the valve approaches three-quarters closed the flow is cut off rapidly. It is the rate at which this last portion of the closure is made that determines the surge conditions. With a motor driven gate valve operating though its complete stroke in two minutes, it is probable that the last 30 percent of the stroke will cut off 90 percent of the discharge, thus the rate of change of flow is determined by the rate

of valve movement during this last portion of the closure, and the closing time should be based upon a value which would cause this portion of the stroke to take place at a rate sufficiently slow as to avoid any dangerous conditions.

It is now possible by means of recent advances in the theory of water hammer to analyze the more important conditions encountered in water works pumping problem and also the water works distribution systems. The Committee on water hammer of the A. S. M. E. is preparing a Second Symposium on Water Hammer which will probably be held in December of 1937, at which time principal emphasis will be placed upon water hammer conditions encountered in water works installations. An invitation will be extended to the American Water Works Association and its affiliated groups to participate in this meeting, so that the problems of the water works industries can be given proper consideration.

THE DISPOSAL OF BRINES FROM OIL DRILLING AND PETROLEUM OPERATIONS*

By Earnest Boyce

(Chief Engineer, State Board of Health)

(Lawrence, Kansas)

We are prone to accept a present abundance of our natural resources, including water, with a careless disregard for the future. It would seem necessary that a shortage occur before public interest and thought are aroused to a comprehension of the need for conservation measures.

Our drought losses will be minimized by any contribution that may be made to constructive thinking in matters of water conservation, and as we count these losses that have accrued due to the drought, we may find compensating values in an increased public appreciation of the need for water conservation. However, we should be warned that this public interest may carry with it an insistent demand, born of the stress of drought, that quick action be taken in the adoption of corrective measures, and that public sentiment may approve certain construction programs before proper geological, engineering, and economic consideration can be given to all phases of the problem.

It must be recognized that a sustained public interest is difficult if not impossible to maintain, and that there may be good reason for striking while the iron is hot and utilizing the advantages of the moment, regardless of the exact pattern that may result or its fitness for the service expected. Contrasted with this is the more deliberate and, may we suggest more economical and engineering-like procedure which would utilize the interest of the moment to secure support for the preparation of plans providing for an orderly procedure in the development and utilization of our resources.

Such plans can come only as the result of careful study of a great mass of basic data pertinent to the problem. Any attempt to trans-

*Presented before the Missouri Valley Section, October, 1936.

late preliminary conclusions into quick sketches for immediate construction cannot avoid at least a partial failure. A program of water conservation can be successful only if it provides for a carry-over from years of ample rainfall in an amount sufficient to give reserves adequate to the necessary demands of—not one dry year—but the most disastrous combination of dry years predictable from a study of rainfall records.

Methods of water storage are properly related to the natural geological and topographical conditions that exist in each locality where water usage makes special conservation measures necessary. It is doubtful whether any method of water storage should be considered as generally adoptable.

In making an inventory of our water resources, we should recognize that the water in storage must not only be adequate in volume but it must also be of such quality as to be suitable for anticipated usage. This requires the adequate control of all sources of organic and mineral pollution that may threaten the quality of water in storage either on the surface or in the ground.

Planning for water storage should have coördinated with it a comprehensive analysis of all the factors that may now or in the future modify the quality of the water in storage. A failure to recognize the importance of solving the problems of waste disposal as a part of a water conservation program may, in some areas, defeat the purpose of such a program.

FACTORS AFFECTING WATER QUALITY

The quality of surface water passing through a drainage basin is dependent on natural geologic factors modified by the development of the watershed by cultivation, deforestation and other changes of the soil covering that tend to increase the soluble and insoluble mineral matter carried by the stream. The quality is also dependent on the character of the liquid organic and inorganic waste incident to the development of the valley. Adequate stream sanitation should reduce to a minimum the permanent effect of the organic wastes reaching the stream. There is no cause to question or discuss the effectiveness of a properly designed and operated modern sewage-treatment plant to so change the character of the organic wastes reaching the plant that a reasonable reuse of the water in the receiving stream is possible.

OIL FIELD BRINES

This paper is concerned with the problem of disposal of inorganic liquid wastes incident to oil production, commonly referred to as oil-field brines. These wastes are, for the most part, chemically and biologically inert, but because of the concentration of mineral salts present, their discharge into streams that must provide storage to serve down-valley cities and industries creates a serious problem of water-supply contamination.

Because of the mineral nature of these brines, they do not change except in intensity of concentration. Unless there is water available for adequate dilution the concentration of mineral salts, principally the chlorides of calcium, magnesium and sodium, may become great enough to destroy the usefulness of the stream for purposes other than waste disposal. Confined to ponds, these wastes, due to their seepage into ground-water formations, often threaten the quality of this important storage.

In order to understand the problem of brine production and to visualize the geological conditions which make its control difficult, this paper will review briefly the occurrence of brine in a producing oil field.

OCCURRENCE

The brine is pumped to the surface with the oil and becomes a waste product when separated from the oil by gravity in field tanks. The brine brought to the surface with the oil may have its origin in the rock overlying the impervious formations against whose under surface the oil and gas have collected, or it may come from the formations that underlie the oil horizon, or it may be a mixture of both top and bottom water.

The significance of oil-field brine is indicated by the fact that most wells are abandoned while they are still capable of producing some oil, but due to the increased proportion of brine to oil pumped to the surface further operation ceases to be economical. The economic limit of the oil to water ratio that will permit continued operation depends on the cost of maintenance and pumping, the cost of brine separation and disposal, and the value of the crude petroleum produced.

Petroleum geologists have estimated that by the commonly used methods of flowing and pumping, less than half of the oil is recovered from producing formations. The subsurface movement of oil-field

brines plays an important part in controlling the life of a producing field. Movements of salt water are created by an unbalancing of pressure due to the removal of gas, oil and salt water incident to the development of the field. Movement must of necessity be toward areas of reduced pressure, i.e., the area of oil-producing formation. The path of travel may be either through natural faults, fissures, solution channels, openings made by drill holes, or through the interstices between sand grains displacing the oil in the direction of the reduced pressure area centering about the well being pumped.

Due to the lesser specific gravity of oil as compared with salt water it has a tendency to move upwards toward the surface of the ground until stopped by gas-locked passages or impervious formations. Hence, it is logical to presume that in general no natural openings exist for the escape of oil or the entrance of salt water except on the edges of the impervious cap rock below the bottom of the oil-water horizon. The migration of salt water from formations of higher to those of lower pressure, through imperfectly cased producing wells or carelessly plugged abandoned drill holes, is guarded against by laws and by the coöperative effort of those oil companies that appreciate the dangers of such salt-water migration.

Migration of brines from higher formations to lower oil-producing formations may be expected to occur naturally in most fields as the oil is removed from below the impervious rock. It is important to note, however, that it must enter the oil-producing formation below the original oil-water horizon if it enters through natural passages. Failure to confine top-water formations during the drilling of a well may result in a movement of the water downward to the bottom of the well, permitting it to enter the casing with the oil. Excess water entering the oil formation through a well will tend to flood the oil formation, thus reducing the volume of available oil, and may become a primary cause for abandoning the well. Improved methods of well-drilling, including the cementing of the annular space between the casing and the drill hole, have greatly reduced this salt-water hazard in properly drilled new wells, and to a limited extent have made possible the reconditioning of existing wells resulting in an increase in oil production and a decrease in the salt water.

The second source of salt water in a producing well is from the formation underlying the production horizon. This may be caused

by the drill hole penetrating too far into the oil-producing formation, making it possible for the bottom water to enter the well with less resistance than the oil, or by pumping at a rate greater than the rate of oil movement to the well, thus causing the bottom water to rise (usually referred to as "coning"). In the first case, bottom plugging has been useful in reducing the volumes of water pumped and in the second case a reduced pumping rate is indicated. Unfortunately the source of water may be difficult to trace. Brines that have had long contact with oil formations may have distinctive modifications that identify them from water coming from top formations. This modification is usually indicated by a marked reduction in sulphate content as compared with other deep-seated waters in the same area, a change that has been attributed to a reducing action of the petroleum compounds. Regardless of the salt water that may enter a well due to imperfect construction or improper operation, in most fields bottom water may be expected to rise under hydrostatic head as the oil is pumped and eventually reach the well as the center of the reduced pressure in the oil formation. Therefore increased salt-water production may be anticipated in most oil fields as the available oil is depleted and its appearance does not of necessity indicate improper development. From this brief review it is evident that most oil-producing areas sooner or later have to solve the problem of oil-field-brine disposal.

The character of the mineral salts contained in the waste water from oil wells makes it impossible to modify these brines chemically to a point where the water would be usable. Since the salts are objectionable largely because of their concentration, dilution with fresh water is a possible method of disposal.

DISPOSAL METHODS

Satisfactory disposal by dilution requires that there must be adequate flow in the stream whenever wastes are discharged. There are two ways of doing this: First, by holding the wastes in reservoirs until the natural flow of the stream will give adequate dilution, and second, by increasing the dry-weather flow of the streams by a controlled discharge of fresh water impounded above the source of these mineral wastes.

The big difficulty in the problem of releasing mineralized waters stored during periods of low flow is the lack of adequate control authority to give permission to discharge. There must be definite

information available regarding river stages in order to know how much mineral waste can be disposed of by dilution. Stream-flow measurements and rainfall data must be obtained and made immediately available to some responsible authority whose duty it would be to control the discharge of these wastes.

The amount of water available in the average Mid-Continent stream draining oil-field areas during the periods of low flow does not approach the amount required to provide adequate dilution to the oil-field brine produced. This condition first becomes evident in upper tributaries where channels are dry except for storm-water flow. The failure to secure adequate dilution from natural run-off is most acute in the immediate vicinity of a producing oil field, but as a period of low rainfall and runoff continues the diluting effect of the flow from the larger drainage area fails to reduce the concentration downstream to potable limits. Further, channel storage in the streams near the oil fields becomes filled with an increasing concentration of brines. If this concentration of brines is removed during quick, heavy flood a dilution may be secured that will avoid complaint. If the flood is slight it may displace the channel storage downstream without adequate dilution and give cause for complaint at points a considerable distance from the oil-field source.

In order to make use of the dilution effect of flood water and thus avoid complaints and damage suits from riparian claimants using the surface water receiving oil-field drainage, many oil companies have made use of salt-water ponds for storage and evaporation. Where the surface water is to be protected, the use of ponds from the controlled discharge of salt water during periods of flood in the receiving stream would seem to be a satisfactory method of disposal by dilution.

Coöperative agreements between producers make possible the construction of large salt-water storage reservoirs, into which all the salt water produced in the field is pumped and from which it may be discharged, under control, to maintain satisfactory dilution conditions in the receiving stream. While it may be that the total cost of storage in large reservoirs will be more than the cost of storing the salt water on individual leases, the need for an adequate control of the discharge into the receiving stream makes storage in large reservoirs desirable.

While solar evaporation offers a possible method of brine disposal, actual rates of evaporation have not been fully studied and many

operators do not differentiate between evaporation and seepage. Unfortunately, seepage is hard to control and sites for adequate storage are difficult to find and expensive to develop.

ALTERNATE DISPOSAL METHODS

Aside from field evaporation or controlled discharge into adequate volumes of diluting water, two other methods of brine disposal may be considered. First, the evaporation of brines under plant conditions for the recovering of mineral salts; and second, the return of the brines to underground formations.

The use of oil-field brines as a source of mineral salts of commercial value offers possible revenue. Experience with brine evaporation for the production of a commercial product has been tried at Sand Springs, Okla. From available data it would seem that this method is feasible only if the brines contain sufficient amounts of the rarer salts. It is possible that the revenue from the sale of salts recovered from concentrated brines may be used to offset part of the cost of brine disposal, and that the net cost may compare favorably with other methods of disposal.

The return of oil-field brines to geological formations that are normally filled with highly mineralized water is a method of disposal that warrants careful study and consideration and offers the greatest possibility of protecting our fresh water storages.

The removal of oil and salt water from producing formations must of necessity leave the rock openings, formerly filled by the pumped oil and water, filled either with gas or with water that has moved into the space under hydrostatic head. While in some geologic formations there is evidence that the removal of petroleum results in a compacting of the producing formation due to the pressure of overlying formations, there is little evidence that the oil-producing formations of the Mid-Continent area are so compacted. Unless these formations are reduced in volume, it follows that salt water might be returned to the oil-producing sands, for it is obvious that the salt water is of less volume than the salt water and oil pumped from the well. Solar evaporation from ponds might be useful in reducing the volume and increasing the concentration of brines before their return to salt-water formations. If we accept the theory that the brines underlying the oil rise in the structure as the oil and gas are removed, it follows that at some point an equal volume of water might be added to replace

the brine that has moved into the producing formation without disturbing its hydrostatic balance. When the gas pressure of a new field has been exhausted, the water and oil will seek a static level in the well which, when corrected for density difference, is the top of the ground-water table connecting with the bottom water holding the oil perched inversely beneath some impervious formation.

Any geological study of the possibility of underground brine disposal should locate the side of the structure and the input well should be far enough removed that unbalanced water pressure over the producing area would be avoided. While some attempts to return salt water to underground formations have not been successful, there is reason to believe that most of the difficulties encountered have a workable solution.

A number of companies have been able to salvage wells drilled for oil production but located at the side of the oil-producing structure for brine input use.

FURTHER STUDY NEEDED

Further investigation of this method of handling salt water is being made by those having a thorough knowledge of the movements of underground waters as related to the geology of each particular field investigated.

When brine input wells are properly located and constructed primarily for that purpose it is reasonable to suppose that the results obtained will be greatly improved.

In seeking a solution of the problem of brine disposal, we should study the situation existing or anticipated in each field and apply the methods of disposal that are best adapted to that particular area. Failure to plan for the disposal of anticipated brine production invites expensive litigation and damage suits—and money paid in damages does not restore destroyed water resources.

Fortunately for Kansas water resources, the state legislature has recognized the importance of giving attention to the methods of brine disposal and has provided financial support for research studies of this problem by authorizing a special tax against the petroleum produced in the state. These studies have been favorably accepted by the major operators who recognize that the financial burden of damage suits that threaten the industry can best be avoided by adopting some plan of brine disposal that will prevent damage to the riparian claimants of the water resources of the state.

WHAT BECOMES OF THE WATER*

By H. V. Pedersen

(General Manager, Water Works, Marshalltown, Iowa)

Ever since the development of the first water works system there have been two schools of thought regarding the question of whether it should be considered a function of the water department to know what becomes of the water after it leaves the pumping station. Disciples of one school maintain that the water works superintendent should be concerned only with the development of as much water as the demand requires and that the consumer is entitled to use as much water as desired without restrictions of any sort. The other school maintains that it should be a duty of every water works department to account for as much of the water which passes the master meter at the pumping station as is humanly possible and believes that the more information a water works superintendent has regarding the amount of water used by the consumers and for what purpose it is being used, the better it is.

It is not necessary to go very deeply into the question of accounting for water before one discovers that there have been sane reasons justifying both schools of thought. It cannot be denied that cities located on large bodies of fresh water, such as the Great Lakes, have found it just as advantageous to build mammoth pumping stations and lay large enough water mains to supply all the consumers with all the water they could possibly wish, as it is to go to the trouble and expense of installing and reading water meters. Even though water works engineers may, for one reason or another, favor a system of accounting for water in all cities, it would undoubtedly take a long time to convince the general public in some cities of the advisability of it. On the other hand, cities located in districts where water is either so scarce or so costly to develop that it is next thing to being a luxury, conserve the available supplies and restrict the use of water as a matter of course and have learned from experience that the logical way to control the use of water is

*Presented before the Missouri Valley Section, October, 1936.

to equip the water works system with the best means available for informing the superintendent where and for what purpose the water is used.

The degree to which a city should be concerned with the accounting for water after it leaves the pumping station therefore depends upon the degree to which the available water supply approaches inexhaustability. To apply such a yardstick it is quite evident that every city must know all there is to know about its present and future available source of water supply.

Simple as such a yardstick seems, its application is not as simple as it sounds. Unfortunately most people refuse to be concerned about anything else but the present. The average citizens in the United States are not water conservation conscious and care very little what happens to the country after they get through with it. There are, of course, a few exceptions. We can be fairly certain that the people living in the semi arid districts of the western states appreciate the true value of water and what a dependable source of water means to present and future generations. But take our own Mississippi and Missouri river valley sections. Do the people at large know or are they even interested in knowing that the policy in regard to water for the past fifty years has been that of drainage and that this drainage policy now appears to have serious defects? Rivers have been straightened, ditches have been dug and farm lands have been tiled in an effort to rush the water that falls to the ground in the form of rain, back to the ocean as quickly as possible. Thousands of acres of nature's water holes have been put under cultivation and costly dikes have been built along river banks to confine the water within certain bounds. Much money and effort has been spent in carrying out a water policy which has been based entirely on the assumption that there will always be too much water and the one big job is to get rid of it.

Occasionally one hears the theory advanced that the failure to rain when rain was needed the most during the past several years is due to the past drainage policy. This may or may not be substantiated by scientific data but one thing seems to be certain. Speeding up of the normal rate of run off and the elimination of nature's storage places tends to reduce the amount of water which would normally find its way into the ground. It is self-evident that if the rate of ground water seepage is retarded and the rate of ground water pumpage increased the time will come when the ground water will be depleted.

It appears that unless the states within the Mississippi and Missouri river valleys undo many of the things that have been done in the guise of drainage and begin a program of controlling run off and storing water in made reservoirs under a new conservation policy some parts of our section will be faced with drastic restrictions in the use of water. It cannot be denied that the quantity of water that can be developed in a community will determine the future growth of that community. Progressive civilization cannot exist without water.

After considering the question of accounting for water I have come to the conclusion that the first thing for those to do who care to align themselves with this school of thought, is decide upon and approve the best means and methods of measuring all the water as it is discharged from the distribution system. The second logical step is to perfect a uniform method of collecting and recording data so that results obtained in one city will be comparable with results obtained in any other city.

The majority of cities that are not fortunate enough to be located where there is apparently an inexhaustible supply of water available for years to come, have for a long time been metering water services of paying consumers. Most cities have a system of telling how much water is actually sold and paid for but few cities have been keeping workable records of the salable water. Most water works superintendents have a vague idea of the amount of water that is discharged from the distribution system for which no pay is received except possibly that which is represented in the form of taxes, but reliable data is woefully lacking. A few cities, here and there, have awakened in the last few years to the realization that it may be desirable to know what becomes of the unpaid for water and have been installing meters in public buildings, city parks and etc. A city that is 100 percent metered, that can tell accurately how much water is used for flushing streets, flushing sewers, fighting fires, drinking fountains and etc. is about as rare as a snow ball in Hades. A perfect system of accounting naturally would be one that accounts for all the water discharged from the mains by accurate measuring devices. Plenty of arguments pro and con, could undoubtedly be raised regarding the practicability of using the ordinary type of water meter to measure the water used in putting out fires. It may be true that more practical measuring devices will have to be developed before an accounting system could be perfected and it may have

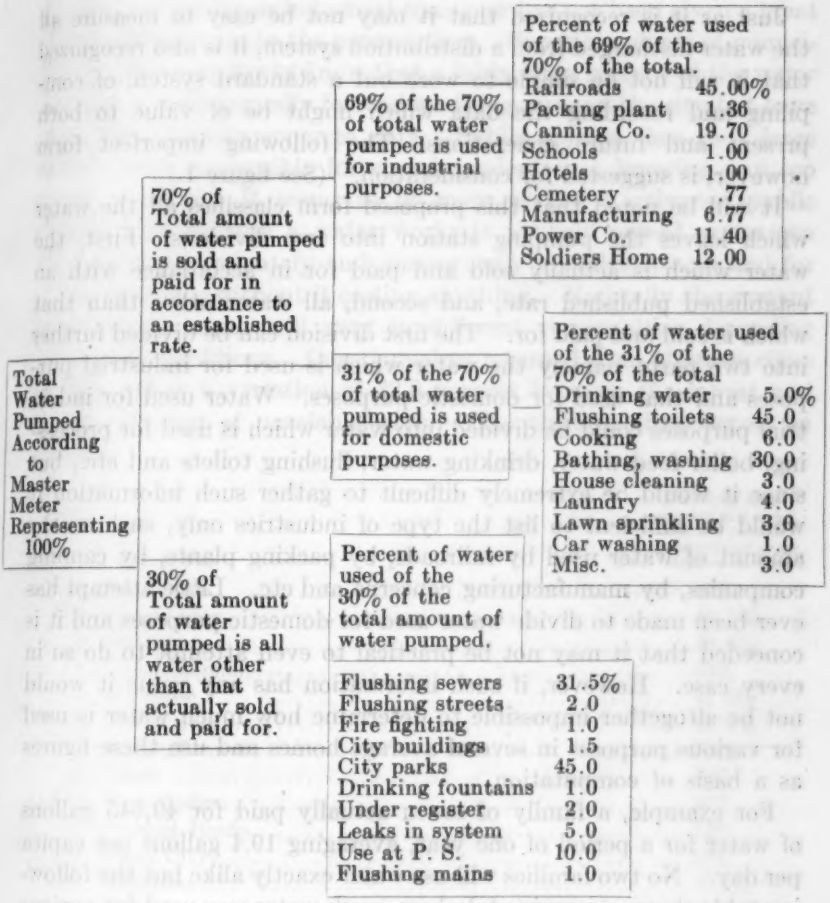

Total Water Pumped According to Master Meter Representing 100%
70% of Total amount of water pumped is sold and paid for in accordance to an established rate.
30% of Total amount of water pumped is all water other than that actually sold and paid for.
69% of the 70% of total water pumped is used for industrial purposes.
31% of the 70% of total water pumped is used for domestic purposes.
Percent of water used of the 69% of the 70% of the total.
Railroads 45.00%
Packing plant 1.36
Canning Co. 19.70
Schools 1.00
Hotels 1.00
Cemetery .77
Manufacturing 6.77
Power Co. 11.40
Soldiers Home 12.00
Percent of water used of the 31% of the 70% of the total
Drinking water 5.0%
Flushing toilets 45.0
Cooking 6.0
Bathing, washing 30.0
House cleaning 3.0
Laundry 4.0
Lawn sprinkling 3.0
Car washing 1.0
Misc. 3.0
Percent of water used of the 30% of the total amount of water pumped.
Flushing sewers 31.5%
Flushing streets 2.0
Fire fighting 1.0
City buildings 1.5
City parks 45.0
Drinking fountains 1.0
Under register 2.0
Leaks in system 5.0
Use at P. S. 10.0
Flushing mains 1.0

FIG. 1

to be admitted that it would be impossible to measure all the water but regardless of all the objections that can be raised against the idea of measuring water there can be no harm at least, in attempting to set up an ideal accounting system as something to shoot at.

Just as it is recognized that it may not be easy to measure all the water discharged from a distribution system, it is also recognized that it will not be simple to work out a standard system of compiling and recording the data which might be of value to both present and future generations. The following imperfect form however, is suggested for consideration. (See figure 1.)

It will be noted that this proposed form classifies all the water which leaves the pumping station into two divisions. First, the water which is actually sold and paid for in accordance with an established published rate, and second, all water other than that which is sold and paid for. The first division can be divided further into two parts, namely the water which is used for industrial purposes and that used for domestic purposes. Water used for industrial purposes could be divided into water which is used for processing, boiler feed water, drinking water, flushing toilets and etc., but since it would be extremely difficult to gather such information it would be sufficient to list the type of industries only, such as the amount of water used by railroads, by packing plants, by canning companies, by manufacturing concerns and etc. Little attempt has ever been made to divide water used for domestic purposes and it is conceded that it may not be practical to even attempt to do so in every case. However, if such information has any value it would not be altogether impossible to determine how much water is used for various purposes in several average homes and use these figures as a basis of computation.

For example, a family of seven actually paid for 49,545 gallons of water for a period of one year, averaging 19.4 gallons per capita per day. No two families will use water exactly alike but the following table shows approximately how much water was used for various purposes by the above mentioned family;

1. Drinking	5%	5. Household cleaning	3%
2. Flushing toilets	45%	6. Laundry	4%
3. Cooking and kitchen work	6%	7. Lawn sprinkling	3%
4. Bathing and washing hands	30%	8. Car washing	1%
		9. Miscellaneous	3%

Applying these percentages to the total amount of water used for domestic purposes would give a general idea of how wasteful the people in a community really are.

Accounting for the water which is not actually sold and paid for by either domestic or industrial consumer has not been given a great deal of thought up to the present time. Superintendents of municipal owned water works know that a fairly large percent of the water pumped is used annually for various things around the city but have very little definite concrete data. Public opinion, has to a large extent, been responsible for superintendents not knowing any too much about it. It seems to be more or less a universal public opinion that because a water works is publicly owned, water can be used indiscriminately and unreservedly in public places and for public purposes without it costing anything. Naturally the amount of unsalable water will vary in different communities depending upon local conditions. Questionnaires returned from thirteen cities in Iowa show a variation of 33.6 percent between the lowest and highest amount of unsalable water. The thirteen cities report as follows.

Name of City	*Percent of water pumped that is not sold and paid for in accordance to an established rate.*
1. Oskaloosa	42%
2. Mason City	41.5%
3. Ottumwa	35.6%
4. Cedar Rapids	34%
5. Dubuque	33.8%
6. Des Moines	31.9%
7. Marshalltown	31%
8. Ames	22%
9. Fort Dodge	18%
10. Council Bluffs	18%
11. Sioux City	15%
12. Waterloo	8.4%
13. Boone	5.68%

Just why there is such a wide variation in these percentages is not for me to say. It is possible that the questionnaire was not quite clear and that incorrect figures were supplied. They may be correct.

At any event the figures indicate that all water works men might do well to pay a little more attention to this item of unsalable water. It is possible that meters on all public buildings and a more careful

check of the amount of water used for fighting fires, flushing streets, flushing sewers, public parks, public drinking fountains, amount used at the water plant, under registration of meters and loss due to leaks would produce results of a different aspect.

I have also pointed out that information which water works men could collect and record regarding the amount of water used for various purposes in each community might be of inestimable value to framers of future water conservation policies. Too little effort has been made, up to the present time, to account for all the water which leaves the pumping station and the records that have been kept are not comparable due to lack of a uniform system of accounting.

The American people have become accustomed to using a great deal of water. However, it has been pretty well demonstrated that most people can get along with a lot less water than accustomed to when a shortage of water has forced restrictions. But is the restriction in the use of water what the people want? If it is, then surely it is high time for water works men to determine, as accurately as possible, just what the minimum water requirements of each community are. If we take a stand on the negative side of the question, can we then sit idly by without at least protesting against a policy that tends to destroy many of the sources of water supply of the Mississippi and Missouri valleys. Would it not be far better for us all to promote a more complete water conservation policy, backed up by data obtained from a systematic study of every phase of the problem. Would it not be well for water works men to contribute their bit to this study by beginning a program of accounting for all the water which leaves the pumping station and to assist in so far as possible in framing a new water policy which will not only insure an adequate public water supply for future generations but will increase the volume of water needed for all other purposes to such an extent that our states can in future years develop into truly the garden spot of the world.

ACTIVATED CARBON—ITS MANUFACTURE AND STORAGE*

By J. P. Harris and E. A. Sigworth

(*Water Treatment Engineers*)

(*Industrial Chemical Sales Division*)

(*West Virginia Pulp and Paper Co., New York*)

WHAT IS ACTIVATED CARBON

Back as far as the 18th Century it was known that charcoal would adsorb gases, and that color could be removed from liquids by it. However, the capacity of charcoal was so small that little advantage was taken of this discovery. It was not until the present century that sufficient progress was made to enable the adsorptive property of carbon to be utilized economically.

We now know that the result accomplished with charcoal was due to a concentration of the impurities at the carbon surface, since these bodies tend to move from the interior toward any exposed surface. It was therefore natural to attempt to make charcoal more effective by tremendously increasing the surface of the carbon particle for exposure to the solution. The success of this, resulted in making the charcoal more active in removal of impurities, and thus the name activated carbon.

It will therefore be realized that the value of activated carbon is dependent upon its physical structure. A particle of good activated carbon can be compared to a very small black sponge where the pores offer a tremendous internal as well as external surface for exposure to the liquid. On the other hand a particle of charcoal can be compared to a piece of coal with practically no internal surface, and what pores are present are loaded with hydrocarbons, leaving no open spaces for the adsorption of impurities.

Most of the commercial qualities of activated carbon appear in the form of a fine black powder. These carbons will vary in appear-

*Developed from a paper presented before the New York Section, December, 1936.

ance and chemical composition. It is therefore impossible to determine by any of the common methods of analysis, what is and what is not activated carbon. They are only recognized by their ability to adsorb or remove from solution undesirable impurities. Therefore, an activated carbon may be described as a material which will remove such materials as tastes and odors, colors, or other impurities from solution. They will usually differ from an ordinary charcoal only in their physical structures which unfortunately cannot be seen even under a powerful microscope. The surface of a particle of activated carbon is tremendously increased during the process of manufacture, to such an extent that it has been estimated that one cubic inch of a good activated carbon will have an external and internal surface of over 20,000 square yards. The fineness of a carbon often plays a tremendous part in the adsorptive action. One gram of a good activated carbon has been reported by a reputable microscopist to contain 120 billion particles. However, we have had cases where the number of particles in an activated carbon was only 20 billion per gram.

MANUFACTURING METHODS

Activated carbons work by adsorption, attracting to their tremendous surface impurities present in the solution. These are held very tenaciously, and are removed from solution with the carbon during the filtration process. If the activated carbon particle has received proper treatment, so that the surfaces exposed are entirely clean, the adsorption will naturally be much greater.

In the manufacture of activated carbon this result is accomplished primarily by closely controlled burning. Almost any vegetable substance may be utilized as a raw material, namely, wood, charcoal, peat, coal, lignite, etc. If necessary the raw material is first converted to a char, prior to the activation stage. It is then heated to definitely controlled temperatures in the presence of such gases as carbon dioxide, air, steam, chlorine and the like. Occasionally chemical reagents are employed, such as phosphoric acid, zinc chloride, sulfuric acid, lime and chalk. In any case it is essential to closely control the gases and temperature to insure a uniformly good quality of product. The carbon after activation is very finely ground so that when shipped it will be almost completely 300 mesh material or finer. It is then packed, usually in strong kraft paper bags, cartons, or drums, and stored ready for shipment.

SHIPMENT OF CARBON

There are no restrictions as related to fire and other hazards in the shipment of activated carbon. Interstate Commerce Commission regulations do not permit bulk shipments under the present ruling and, therefore, take into consideration only the strength of package. Those packages which meet the requirements are four ply multi-wall paper bags, cloth bags, barrels or boxes. Due to the fact that activated carbons are very porous in nature, and therefore very light, the packaging cost per pound is a very considerable item. Consequently, it is the usual practice to avoid the more expensive type packages, such as barrels and boxes.

With the increasing use of activated carbon for taste and odor control in municipal water plants, sufficient material is now being used in many cities to warrant taking in carload shipments for storage. Considering the differentials between carload and less carload freight rates, it is felt that it would be economical for any city utilizing a carload or more per year, to make arrangements for storing so that a carload can be placed in stock. With the improved business conditions at the present time, making more funds available for expansion, it is safe to say that many cities will consider the erection of proper storage facilities for activated carbon.

GENERAL PRECAUTIONS IN STORAGE

In more than twenty years of producing and marketing, in the industrial field, the carbon most widely used in water purification, we have never known of an explosion occurring from this carbon dust or from its spontaneous combustion. Nevertheless it is a combustible material and, being supplied for water treatment in a very finely divided state, it is likely to fly about and penetrate machinery, causing damage from the blocking of lubrication and from ignition, particularly in electrical equipment.

Also it should be borne in mind that it has such a high activity for the adsorption of foreign materials that it is possible to generate heat during storage in the dry condition, and, for this reason, it is obvious that it should not be stored in a chlorine room, where the adsorption of chlorine due to a possible leak might generate enough heat of adsorption to reach the ignition temperature.

In view of this, activated carbon should never be mixed in a dry state with hypochlorite of lime, since to do so may cause a violent explosion.

Carbon should not be stored where oil or gasoline can come in contact with it, because it is a very porous material as well as a good heat insulator. Such materials may easily soak to the inside of a body of carbon where the heat developed by the slow oxidation of the oil may be retained and accumulated until the ignition temperature is reached.

Activated carbon should be stored in a room or building completely isolated from its point of application, and in such a location in the building that it is not close to stoves or other direct heating devices, machinery in operation, or electric switches and/or wiring which are not completely protected.

The stored carbon should be piled upon racks, preferably of metal construction, so as to keep it two to four inches off the floor, thus permitting a free circulation of air around and under the piled carbon at all times, and protecting the carbon from water in case of accidental wetting of the floor from any cause.

Regular inspection of roofs over storage should be maintained to guard against flooding the carbon from above.

Avoid all flames in the room where the carbon is stored. "No smoking" signs should be posted prominently in the storage room, and at the points of application.

Where practical, it is advised that an overhead sprinkler system be installed both in the storage and feeding room. It should be emphasized at this point that water in carbon does not destroy its activity. However, any wetting of the carbon would give trouble in applying by a dry feed machine. Therefore, in the event the sprinkler system lets go at any time it is suggested that prompt attention be given the carbon in storage to eliminate any excessive wetting.

The floor, at the point of storage and use, should be properly pitched and drained, so that the accumulated dust may be easily and quickly washed away.

Preferably, only the amount of carbon which will be used within a very short time should be stored at the point of application and this should be carefully protected against fire hazards, and against excessive dusting.

PRECAUTIONS WITH REGARD TO ELECTRICAL EQUIPMENT

We refer you to the "National Electrical Code" regulations of the National Board of Fire Underwriters for Electric Wiring and

Apparatus as recommended by the National Fire Protection Association, Article 32, covering "Hazardous Locations," "Class II, Group F, Atmospheres containing carbon black, coal or coke dust," and "Class II; Group G—Atmospheres containing grain dust."

Class II section (2) locations are those where it is impracticable to prevent such combustible dust from collecting in such quantities on or in motors, lamps, or other electrical devices that they are likely to become overheated because normal radiation is prevented.

While this does not specifically cover activated carbon, there is a sufficient similarity between carbon black and activated carbon to warrant our recommending the use of Electric wiring and apparatus as recommended under Class II, Group F of "the Code."

Through the courtesy of the National Board of Fire Underwriters we quote from "The National Electric Code," "Class II Locations," as follows:

"Service entrance equipment, and all panel boards and switchboards should not be placed in locations of this class, but, if impracticable to locate elsewhere, all live parts shall be enclosed in dust-tight metal cases or cabinets having provision for external operations only, or shall be located in a separate dust-tight room built of or lined with substantial non-combustible materials and having a self-closing door, so constructed and installed as to adequately exclude dust."

"Fuses shall not be placed in locations described in this class unless enclosed in dust-tight metal cabinets or cases. Circuit-breakers shall be of the dust-tight or dust-tight oil-immersed type."

"Rigid conduit shall be the wiring method employed. Where it is necessary to employ flexible connections, as at motor terminals, a short length of flexible steel conduit may be used. Conduit boxes and other fittings used in locations where dusts are of an electrically conducting nature shall be of the dust-tight type."

"Where explosive dusty atmospheres are likely to be present, . . . motors or generators having brushes or sliding contacts shall have such brushes or sliding contacts enclosed in substantial dust-tight housings or shall be of the totally enclosed, totally enclosed-fan-cooled, or totally enclosed-pipe-ventilated types or shall be enclosed in separate dust-tight rooms or housings, built of or lined with substantial non-combustible materials and so constructed as to adequately exclude dust, properly ventilated from a source of clean air."

"It is recommended that all motors installed in dusty locations be

of the totally enclosed, totally enclosed-fan-cooled or totally enclosed-pipe-ventilated types."

"Motor controllers, thermal cutouts, switches, relays, the switches and contactors of auto-transformer starters, resistance or impedance devices, or other devices or apparatus, which in their normal operation tend to create arcs, sparks, or high temperatures, shall not be installed in these locations unless such devices or apparatus are enclosed in dust-tight cases or cabinets or are of the dust-tight or dust-tight oil-immersed type, and so designed that the device may be operated without opening the cabinet or case."

"It is recommended that motor control devices be of the remote control type with the main contractors located in separate rooms of compartments built of non-combustible materials so constructed as to exclude dust. The remote control push buttons or swtiches of the oil-immersed or dust-tight type may then be placed in the most convenient locations."

"Electric heating appliances shall not be used unless of a type approved for installation in dusty atmospheres." (See drawing of an approved type now in use at a large midwestern purification plant. Fig. 1.)

"Lamps in fixed positions shall be enclosed in approved dust-tight globes, and where exposed to mechanical injury, shall be protected by substantial guards. Heavy fixtures used as pendants shall be supported by conduit hangers or chains to prevent strain on the wires. Where rubber-covered wire is used it shall have insulation not less than $\frac{3}{64}$ inch thick. Sockets and receptacles shall be of the keyless type, and unless individual switches are provided, shall be located at least $7\frac{1}{2}$ feet above the floor, or shall be otherwise so located or guarded that the lamps cannot readily be backed out by hand."

"When light is required for the interior of bins, hoppers, elevators, conveyors and similar equipment or construction, it is recommended that such light shall, whenever practicable, be supplied by means of lamps, enclosed in dust-tight globes properly protected against mechanical injury, and mounted flush in the walls or floors of the equipments or construction. No wiring or fixtures should be permitted inside of bins, hoppers, elevators or conveyors."

"Sufficient general illumination shall be provided by fixed lighting units to eliminate so far as possible the need for portable lamps. When portable lamps are necessary, they shall be enclosed in dust-

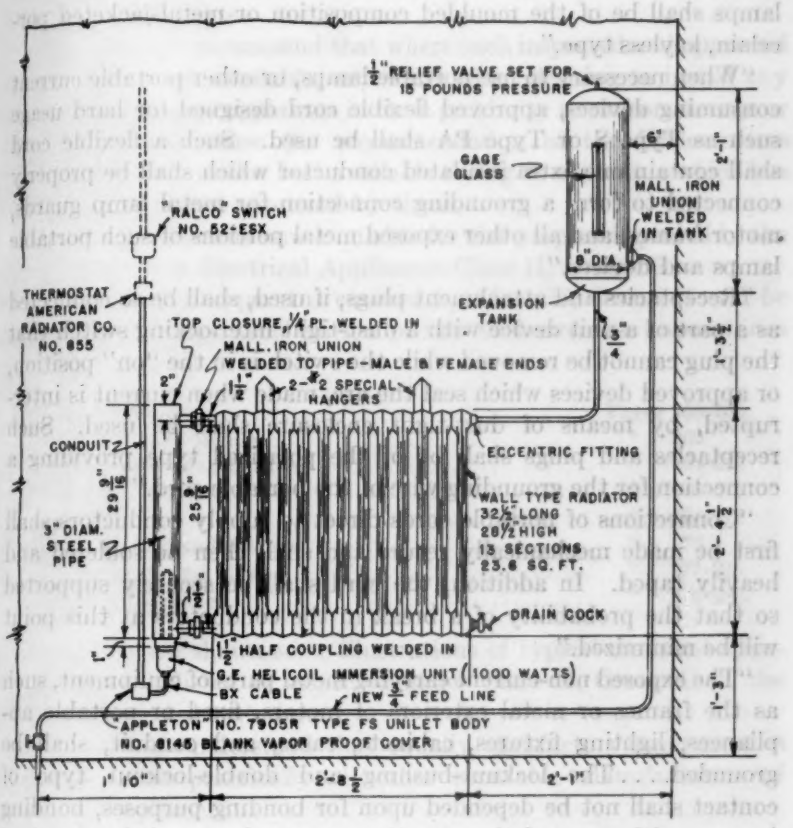


FIG. 1. ELECTRIC HEATER ASSEMBLY FOR CARBON FEEDER ROOM

This arrangement is designed to avoid the use of stoves, open heating coils, switches, etc., in small isolated buildings where central heating is not available. Everything is shown on the drawing that would be needed by a man who wished to duplicate the installation. This heating unit, radiator, etc., are installed in a brick building of 337.5 cubic footage with exposure on 4 sides.

tight globes properly protected by substantial metal or other approved types of guards to prevent breakage. Sockets for portable lamps shall be of the moulded composition or metal-jacketed porcelain, keyless type."

"When necessary to use portable lamps, or other portable current consuming devices, approved flexible cord designed for hard usage such as Type S or Type PA shall be used. Such a flexible cord shall contain one extra insulated conductor which shall be properly connected to form a grounding connection for metal lamp guards, motor frames, and all other exposed metal portions of such portable lamps and devices."

"Receptacles and attachment plugs, if used, shall be so connected as a part of a unit device with a dust-tight interlocking switch that the plug cannot be removed while the switch is in the "on" position, or approved devices which seal the arc, made when current is interrupted, by means of dust-tight enclosure shall be used. Such receptacles and plugs shall be of the polarized type providing a connection for the grounding wire of the portable cord."

"Connections of portable cords direct to supply conductors shall first be made mechanically secure and shall then be soldered and heavily taped. In addition, the cord shall be securely supported so that the probability of a break in the conductors at this point will be minimized."

"The exposed non-current carrying metal parts of equipment, such as the frames or metal exteriors of motors, fixed or portable appliances, lighting fixtures, cabinets, cases, and conduit, shall be grounded. . . . The locknut-bushing and double-locknut type of contact shall not be depended upon for bonding purposes, bonding jumpers with proper fittings or other approved means being required to assure an effective grounding circuit."

"There shall be no exposed live parts."

We regret very much that the "List of Inspected Electrical Appliances of the Underwriters Laboratories" of the National Board of Fire Underwriters does not include any motors, circuit breakers, light fixtures and receptacles and plugs, approved for Class II, Group F, outside of a limited range of circuit breakers which are manufactured by the Crouse Hinds Electric Co. of Syracuse, N. Y. These are listed on page 258 of the May, 1936 issue of the Underwriters Laboratories, Inc. "List of Inspected Electrical Appliances."

It is to be hoped that the manufacturers of electrical apparatus will see that such inspected and approved apparatus is made available at an early date.

Meantime we recommend that where such inspected and approved equipment is not available, any water plant desiring to install any electrical wiring or apparatus in close proximity to carbon storage or feeding should secure the recommendation of the Underwriters Laboratories as to the best available wiring or apparatus which has been inspected and approved by them.

The best apparatus now available for the purpose is probably the list of Inspected Electrical Appliances Class II, Group G.

This inspected apparatus covered by Class II, Group G will be found in the May 1936 edition of the Underwriters Laboratories list, as follows:

Circuit Breakers	Pages 259–260
Conduit and Fittings	" 262
Electric Lighting fixtures	" 264–265
Auxiliary Devices	" 270–271
Motors	" 274–275
Outlet boxes and fittings	" 275
Receptacles and plugs	" 277–278
Enclosed switches	" 279–280

Following are shown some illustrations of typical electrical apparatus which is listed as inspected under Class II, Group G of the Underwriters Laboratories.

It should be understood that equally satisfactory equipment is available from practically all other reputable manufacturers of electrical equipment.

PRECAUTIONS WITH REGARD TO CHEMICALS

(a) Carbon should never be stored where it has an opportunity to come in contact with gases or vapors such as chlorine, sulfur dioxide, gasoline, exhaust fumes from diesel engines, etc. These fumes not only constitute a fire hazard, but may also decrease the efficiency of the activated carbon by being adsorbed upon its surfaces. Activated carbon retains its full adsorptive ability indefinitely if stored under conditions where it is impossible for it to take on foreign vapors. Storage in a merely humid atmosphere does not adversely affect its activity.

(b) It is safe to mix dry activated carbon with alum, soda ash,

hydrated lime (not quick lime), or any other non-oxidizing chemical, but not with hypochlorites or any other chemical which gives off corrosive vapors.

PHYSICAL PRECAUTIONS

Carbon should be stored in a dry clean place, preferably on a slightly raised platform, where oil, water and other materials which may accumulate on the floor cannot come in contact with and damage either the carbon itself or its containers, etc.

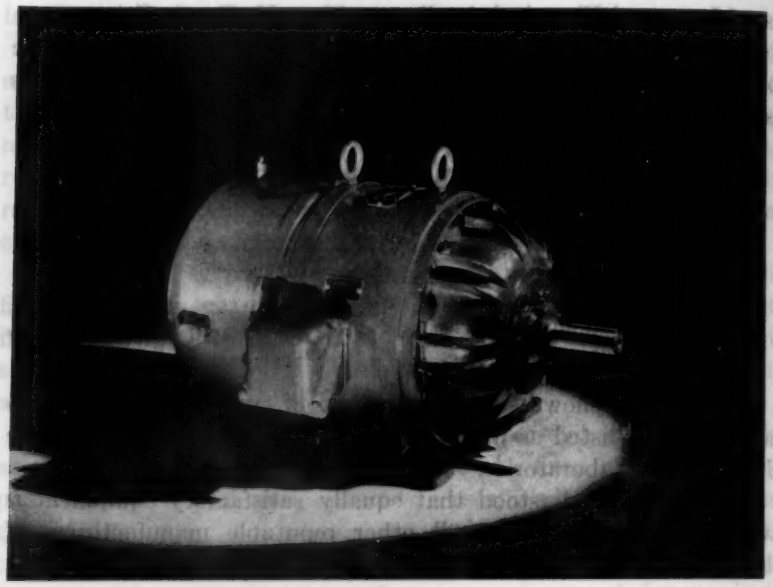


FIG. 2. GENERAL ELECTRIC TYPE K, SLEEVE BEARING TOTALLY ENCLOSED FAN COOLED INDUCTION MOTOR

Fire can only be caused by carelessness and failure to observe the above precautions, but if a fire does occur, the following procedure is recommended:

Do not attempt to extinguish the fire by squirting or throwing water or chemicals on it! Such procedure will cause the very light smouldering particles to fly into the air, thus spreading the fire to unaffected parts of the building. These particles may also settle on the clothes of those endeavoring to extinguish the fire and cause severe burns wherever they may come in contact with the body.

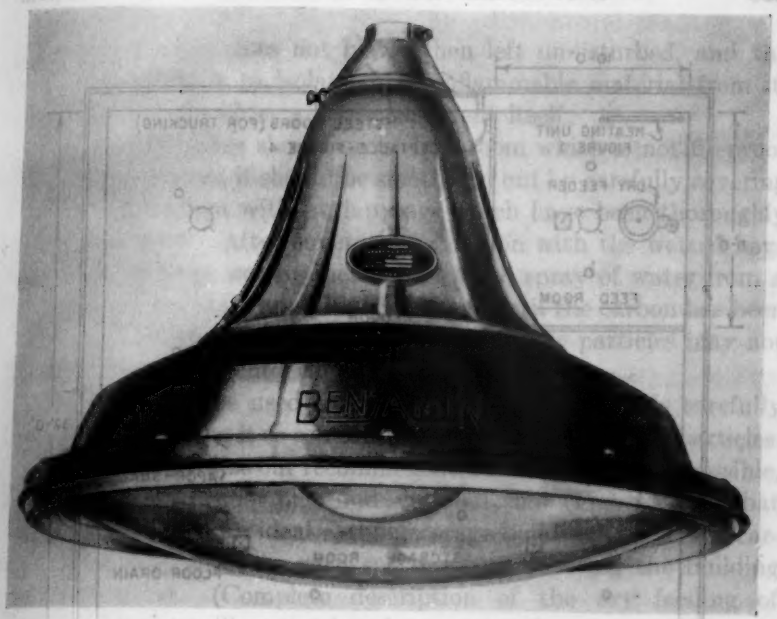


FIG. 3. BENJAMIN VAPORSEAL LIGHT FIXTURE #5650

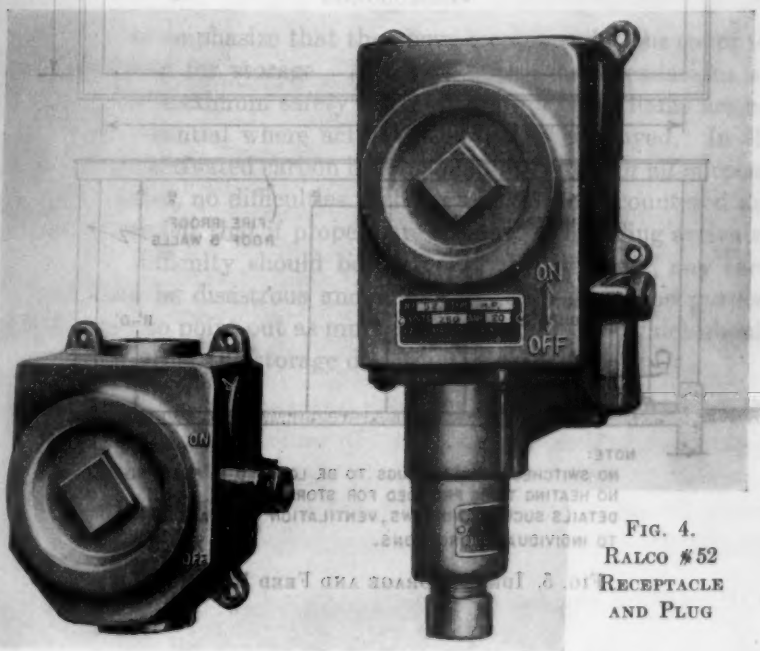


FIG. 4. RALCO #52 RECEPTACLE AND PLUG

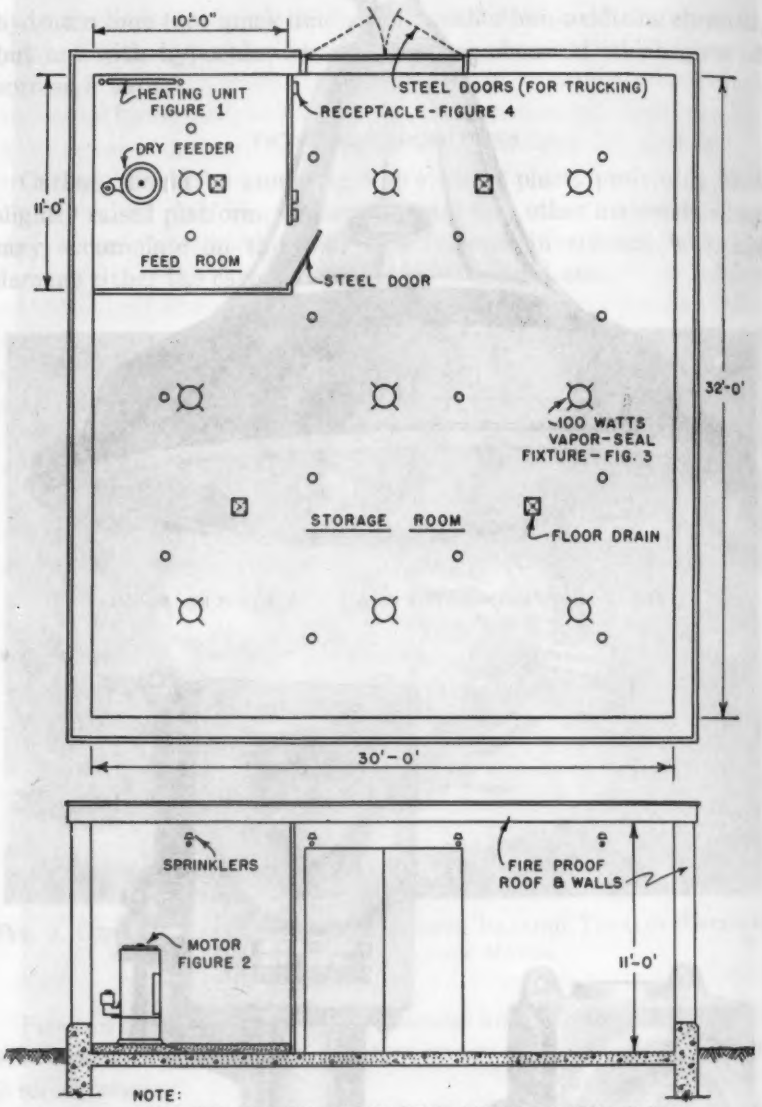


FIG. 5. IDEAL STORAGE AND FEED BUILDING

Burning carbon does not blaze when left undisturbed, and the safest procedure is to isolate other inflammable material from it, allowing it to smoulder until it has spent itself.

In case such a fire should occur in a room which is not fireproof (i.e. wooden floors), it should be smothered out by carefully covering the glowing carbon with burlap bags which have been thoroughly soaked in water. After covering the carbon with the wetted bags they may be kept wet by means of a small spray of water from a hose, but do not start applying this spray until the carbon has been completely covered so that the smouldering fine particles may not be permitted to fly into the air.

Sand may also be used to smother the carbon if it is carefully applied by shoveling it so as not to disturb the smouldering particles.

In order to visualize our recommendations as definitely as possible, we show herewith floor plan and sectional view (Figure 5) of what may be considered an ideal carbon storage building to house a carload of carbon, together with specifications covering the building and equipment. (Complete description of the dry feeding of activated carbon will appear in a future paper.)

CONCLUSION

We wish to emphasize that the above recommendations cover the ideal condition for storage. Although such recommendations are advisable for maximum safety in storage, all of these items are not necessarily essential where activated carbon is employed. In the plants where activated carbon is being manufactured for all purposes including water, no difficulties with fire have been encountered and it is therefore felt that if proper care is taken in handling activated carbon, no difficulty should be encountered. However, any carelessness may be disastrous and it has therefore been the purpose of this paper to point out as much as possible the care which should be utilized during the storage of activated carbon.

DETERMINATION OF THE DENSITY OF POWDERED ACTIVATED CARBON*

By Edw. S. Hopkins

(Principal Sanitary Chemist, Bureau of Water Supply)

(Department of Public Works, Baltimore, Maryland)

The occluded air in powdered activated carbons passing 200 mesh sieve prohibits the use of ordinary pycnometer methods for density determinations. It causes a small portion of the material to remain in suspension making it impossible to read the meniscus correctly. Immersion of these materials into a non-solvent liquid and submergence for a definite period in boiling water, will drive out the entrapped air and permit accurate volumetric readings of the pycnometer. A somewhat similar method to determine the density of coke, using benzene as the immersion liquid, was adopted by Rose (3).

PROCEDURE

Accurately weigh a 25 gram sample, the moisture content of which has been previously determined, into a 250 cc. beaker and thoroughly wet with a small quantity of water. Dilute this paste with an additional 25 cc. of water to the consistency of a thick slurry. Immerse the beaker into *rapidly* boiling water with occasional vigorous stirring for 10 minutes followed by constant stirring for another 5 minutes, a total heating period of 15 minutes. The adsorbed air is driven off, forming a froth at the surface of the liquid, which is dispelled by the stirring. When all of the air has been expelled, the carbon changes in appearance from a dull to a shining black mirror. The sample is then washed into a standard Le Chatelier cement flask and allowed to cool at room temperature, or at 20 degrees Centigrade, as was done in these experiments. By calibration of the flask with a balance of sufficient capacity (500 grams) and sensitivity (0.1 gram), the density of the material can be calculated by the usual formula:

*Contributed record of research.

$$\text{Density} = \frac{a}{a - (b - c)}$$

when a = weight of powder in air
b = weight of powder and water
c = weight of water

For conversion to a dry basis, the moisture content is determined by heating at 110 degrees Centigrade for 12 hours.

RESULTS OF TESTS

As shown in Table 1, six commercial carbons and four other powdered materials were tested by this method, giving density readings of reasonable accuracy.

TABLE 1

MATERIAL	NUMBER OF TESTS	DENSITY		
		Maximum	Minimum	Average
Carbon "A"	10	1.8692	1.8230	1.8492
Carbon "B"	10	1.8511	1.8019	1.8312
Carbon "C"	3	2.1778	2.1200	2.1478
Carbon "D"	3	1.8251	1.7979	1.8070
Carbon "E"	2	1.8598	1.8284	1.8441
Carbon "F"	3	1.6766	1.6543	1.6669
Talc	2	2.6738	2.6738	2.6738
Kaolin	2	2.6208	2.5938	2.6073
Fullers Earth	2	2.5601	2.5315	2.5458
Infusorial Earth	3	2.2997	2.2775	2.2923

These values are in agreement with those obtained by Cude and Hulett (1) of 1.844 and 1.888 for a powdered steam activated cocoanut carbon when dispersed in water. Harkins and Ewing (2) also obtained density values of 1.843; 2.112; 1.808; 1.863; 1.835; 1.934; 1.876; 1.644 and 1.499 for various granular activated carbons. As suggested by these authors, these data cannot be considered as absolute densities, in that different solvents give considerable variation in results, more or less proportional to their ability to penetrate into the micro pores of the material. However, for the purpose of writing purchase specifications, these values are of sufficient accuracy for practical purposes.

REFERENCES

(1) CUDE, H. E. AND HULETT, G. A., Jour. Amer. Chem. Soc., 42, 391 (1920).
(2) HARKINS, W. D. AND EWING, W. W., Ibid., 43, 1787 (1921).
(3) ROSE, H. J., Ind. Eng. Chem., 14, 1047 (1923).

TASTE AND ODOR PROBLEMS*

By William T. Bailey

(Chemist, Water Department, Council Bluffs, Iowa)

So much attention has been directed to the problem of producing a more palatable water during the past decade, and the progress along this line has been so great that it is now possible under most circumstances either to remove or prevent the formation of taste and odor producing substances sufficiently to give a very satisfactory product. There is so much literature available on the subject that it should be unnecessary to do more than outline very briefly the causes and remedies of tastes and odors before proceeding to our own experience along this line.

GENERAL OBSERVATIONS

The causes of tastes and odors have been grouped into the following general headings by R. A. Irwin (1):

1. The presence of organisms and vegetable matter in the raw water.
2. Industrial wastes and domestic sewage.
3. Dissolved gases such as hydrogen sulphide.

Since these classifications all deal with taste and odor producing substances present in the raw water as it comes from the source of supply it becomes necessary to consider a fourth group in the case where the water is retained in open reservoirs for long periods of time, and that is the growth of organisms.

The most common method of treatment usually consists of one or more of the following (2):

1. Aeration
2. Ammonia-chlorine
3. Pre-chlorination
4. Powdered-activated carbon
5. Copper sulphate

*Presented before the Missouri Valley Section, October, 1936.

Naturally the method used must be worked out to best fit the particular plant in question and for the particular water being used. It is possible to get what appears to be very conflicting results if one attempts to duplicate treatment being used very successfully in another plant without making proper allowance for the differences in the physical structure of the plant, even though both are handling water from the same source of supply. And it often becomes necessary to compensate for lack of proper treatment facilities by chemical means.

COUNCIL BLUFFS CONDITIONS

The raw Missouri River water contains some organic matter throughout the entire year with greater quantities periodically. This concentration is usually greatest at the time when the ice goes out of the river and melting snows carry considerable amounts of decaying vegetation into the water, also occasionally when unusually heavy rainfall occurs along some of the tributaries of the Missouri in the farm belt.

Inasmuch as we do not have filters at Council Bluffs, relying entirely on coagulation and sedimentation for clarification, the water is exposed for considerable time in open reservoirs with turbidities varying from 30 p.p.m. down to as low as 1 p.p.m. Our greatest source of tastes and odors comes from algal growths in these reservoirs and from decaying organic matter in the sludge settled out in them. The lack of filters necessitates complete coagulation of the suspended matter in order to produce good tap water, and the entire bacterial load must be taken care of in the process of clarifying and by chlorination.

Our particular taste and odor problem and the steps in its solution may then be summed up as follows:

1. The removal of organic matter from the raw water.
2. Stabilization of sludge in settling reservoirs.
3. Prevention of the growth of algae in open reservoirs.
4. Prevention of bacterial aftergrowths which produce offensive conditions in the "dead ends" of the distribution system.

In a paper presented before the Omaha meeting of this Section two years ago our treatment was outlined, with the statement that the only change contemplated was the addition of ammonia with the chlorine being applied in the open four and one half million gallon reservoir floating on the system uptown (3).

A brief outline of the treatment is as follows:

1. Preliminary settling, 18 hours (total volume displacement).
2. Coagulation with alum and lime followed by 40 hours settling.
3. Addition of powdered activated carbon followed by 8 hours retention.
4. Coagulation with alum and lime, application of sulphate of ammonia and chlorine, followed by 40 hours settling.
5. Addition of anhydrous ammonia and chlorine followed by 72 hours storage, with provision for another application of chlorine at the pumps.
6. Application of anhydrous ammonia and chlorine to the water entering the elevated reservoir floating on the distribution system (the ammonia treatment at this point having been installed in August, 1935).
7. Periodical application of copper sulphate.

The results obtained have been extremely satisfactory with the exception of the period of the "spring run off," when considerable difficulty was experienced in coagulation.

During the summer months the carbon treatment averages about 2 p.p.m., but has been as high as 6 p.p.m. during the "spring run off." It is very helpful in removing organic matter from the raw water, completely stabilizes the sludge in the reservoirs, assists in coagulation, assists in algal control by shutting off their source of light and is very important in the prevention of tastes upon the application of rather heavy doses of chlorine. In general the activated carbon plays a very important part in the production of a clear and sparkling water without the use of filters.

DETAILS OF CHEMICAL TREATMENT

The first application of chlorine is made as the water leaves the mixing chamber from the second coagulation, and after the activated carbon has been in contact with the water for nearly eight hours. The dosage varies from 0.75 to 2.0 p.p.m. with an application of equal amount of ammonium sulphate being made as the water enters the mixing chamber. This treatment is carried so as to give a chloramine residual of 0.50 p.p.m. leaving the settling basins at the end of the 40 hour settling period. As the water enters the clear water reservoirs an application of anhydrous ammonia and chlorine is made in sufficient quantity to give a residual of approximately 0.75 p.p.m. leaving these reservoirs into the distribution

system, theoretically 72 hours later. The treatment at this point is discontinued when the reservoirs are frozen over, and varies from 0.33 p.p.m. to 0.90 p.p.m. chlorine and 0.10 to 0.30 p.p.m. ammonia during the time the reservoirs are not covered with ice. The residual entering the mains is usually kept at about 0.45 p.p.m. during the winter months.

Beginning March 1, 1930, chlorine was applied at the inlet of the elevated reservoir floating on the system in an attempt to control bacterial and algal growths in that reservoir. The operating conditions are such that water enters this reservoir only during a few hours each day. The same main is used as the inlet and outlet for this reservoir. This arrangement permits the application of chlorine into the reservoir during but a few hours each day, and we were unable to maintain sufficient residual in the reservoir to resist the combined action of sunlight and algae from June to October, despite frequent applications of copper sulphate in the reservoir. We were forced to empty and wash this reservoir in July and August nearly every year, and considerable trouble was experienced with "dead ends" in this vicinity.

With the addition of anhydrous ammonia with the chlorine we have been able to maintain residuals of 0.60 to 1.00 p.p.m. within the reservoir this past summer. To do this required 25 pounds chlorine and 8 pounds ammonia daily during the summer months when approximately a million gallons water was being changed daily; and gave good control of the growths when supplemented with copper-sulphate.

Certain attached forms of algae develop a tolerance to chloramine concentrations as high as 1.25 p.p.m. (3). These forms are readily susceptible to copper sulphate. During the summer months we find it necessary to make applications at intervals of one to two weeks. With walls that are perpendicular, or nearly so, the application is made by placing large crystal copper sulphate in a burlap bag and dragging the bag along the wall. But with more sloping walls the small crystal form is placed in a two gallon sprinkling can which has had the spout cut off to give an opening approximately one inch in diameter, and the application made by walking along the reservoir coping and swinging the can in such manner that a continuous line of the crystals are left on the wall just below the water's edge. These dissolve rather slowly and give a very heavy concentration of copper sulphate in direct contact with the algae for thirty minutes or longer; completely eradicating them.

RESULTS OF TREATMENT

The ammonia-chlorine also gives much better bacterial control than we were able to secure prior to its use; completely eliminating the troublesome pin-point colonies appearing in large numbers on plates made at certain seasons of the year. Also this treatment gives much better control of the Colon-Aerogenese groups, there being but two out of a total of approximately six thousand, plantings of 10 ml. portions confirming on eosine methylene-blue agar during the past four years.

With the exception of the period when the ice went out of the river no difficulty has been experienced in controlling tastes and odors during the past year. Beginning March 2, and lasting fifteen days, there was a period when we were unable to remove the suspended and dissolved organic matter as completely as we would like to have done. Successive floods from the Boyer, Little Sioux, Big Sioux and Floyd Rivers brought much colloidal matter which did not respond well to coagulation at the temperature of the water. The color reached 50 p.p.m. at this time and the maximum turbidity was 3000 p.p.m. At the point of coagulation the turbidity maximum was 1900 p.p.m. with the temperature of the water being 34 to 38 degrees F. at this time. By applying Nuchar with the initial coagulation at the rate of 6 p.p.m., and increasing the alum to a maximum of 3 g.p.g. at both points of coagulation some of this colloidal material could not be removed. The first coagulation reduced the turbidity to 300 p.p.m. and the final coagulation left a turbidity of 20 p.p.m., maximum, in the tap water. The tap water carried a slight earthy vegetative taste when cold and a decided earthy vegetative taste when hot. But two complaints were received, both from persons drinking hot water.

Our difficulty at this period was in securing complete coagulation because of the temperature of the water at this time. We feel that by increasing the mixing time and velocity at the second point of application (the present being 10 minutes at 0.5 ft. per sec.) or by coagulation a third time and using two applications of carbon we can control this "spring run off."

An interesting feature of our treatment is that we are able to carry residual chloramine as high as 1 p.p.m. to ortho-tolidene without any so called chlorine taste providing sufficient carbon is being used, even when the water temperature is a low as 40°F.; while tastes re-

sult with residuals of 0.5 p.p.m. and above if activated carbon is not used in sufficient quantity to remove most of the organic matter.

No tastes have appeared in the vicinity of the elevated reservoir when making daily applications of 25 pounds chlorine and 8 pounds ammonia. On the contrary one lady in that vicinity called a few weeks ago to ascertain whether the water was safe to drink, saying that she had not tasted chlorine for so long that she feared that we were not safeguarding the water properly.

Fifty-four reservoir washings were required in 1931 as against thirteen in 1936. Where it frequently was necessary to empty and wash some of the clear water reservoirs as often as four times a year, in order to remove algal growths and putrefactive sludge, the same reservoirs have not been washed since April, 1935. In all probability they will not require washing for at least another year. This reduction in reservoir washing gives a more uniform quality of tap water, and the more uniform operation permits some saving in coagulants.

Formerly we began flushing hydrants as early in the spring as could be done without the water freezing on the streets and continued as late as possible in the fall; flushing all hydrants spring and fall, and all dead ends at intervals of two to four weeks throughout the summer. In 1936 no hydrants were flushed prior to August, and many of the dead ends have not yet been flushed this year.

The savings in labor and water wasted in carrying out these operations, together with lower prices on chlorine and activated carbon when purchased in carlots, largely offsets the increased cost of chemical treatment.

SUMMARY

1. Activated carbon, when properly applied, assists in the removal of organic matter from the raw water, aids in coagulation, stabilizes sludge in reservoirs, aids in the prevention of chlorine tastes, acts as a deterrent to algae and assists in the control of bacterial after-growths in the distribution system by virtue of the removal of organic matter feeding these after-growths, and also by permitting the carrying of higher chloramine residual into the system than would otherwise be possible without disagreeable tastes.

2. Efficient coagulation aids in the removal of taste and odor producing substances. In order to handle properly the "spring run off" it will probably be necessary to secure better floc conditioning in our

plant or perhaps to coagulate this particular water three times. Increasing coagulant dosages may be very important in the removal of some taste and odor producing substances and organisms.

3. Maintaining chloramine residuals of 0.50 p.p.m. or greater in open reservoirs will control the growth of most species of algae, but must be supplemented with applications of copper sulphate in order to give complete control of algae. Such residuals also give improved bacterial conditions both in the reservoirs and distribution system, and, together with activated carbon, does away with much reservoir washing and "dead end" flushing.

4. It is possible to carry chloramine residuals as high as 1.0 p.p.m. without chlorine tastes providing sufficient activated carbon is used to remove a great enough portion of organic matter before the ammonia-chlorine application is made.

5. Increases in chemical costs for taste and odor control may be largely offset by saving in other items such as hydrant flushing, reservoir washing and lower chemical prices. In no event is it justifiable to sacrifice the quality of water in order to effect a saving of a few pounds chemicals.

REFERENCES

(1) Taste Control of Water Supplies. Jour. A. W. W. A., 25: 1202 (1933).
(2) Report of Committee on Control of Tastes and Odors in Public Water Supplies. Jour. A. W. W. A., 25: 1490 (1933).
(3) Taste and Odor Control at Council Bluffs. Jour. A. W. W. A., 27: 458 (1935).

TASTE AND ODOR PROBLEMS AT KANSAS CITY, KANSAS*

By L. B. Mangun

(Chemist in Charge of Water Purification, Kansas City, Kansas)

There is no problem at Kansas City, Kansas, of taste and odor from industrial wastes in the Missouri River. There is no problem of taste and odor due to algae in the river water. Nevertheless there are taste and odor problems, and such as have occurred have been handled with quite uniform success by powdered activated carbon since it was first tried out experimentally in the plant in July, 1931. Aside from prevention of organic growths in the basins and filters, the pre-chlorin-ammonia treatment under the conditions existing at the Kansas City, Kansas, plant is a factor of negligible importance in the solution of these taste and odor problems. The need for eliminative treatment has been intermittent, averaging twice annually during the past five years that activated carbon has been used, and covering a total time of nearly four months out of every twelve.

There have been three major causes of these tastes and odors, two originating in the source of the water supply and one within the plant itself. All are seasonal, when certain conditions are fulfilled. For instance, one may occur in winter, provided the Missouri River freezes over, which happens some years at Kansas City. Taste and odor of the greatest relative intensity that has been met at the plant has occurred when the river was completely covered with ice. But there have been such occasions when no unusual taste and odor was developed. Such was the case last winter when the river was covered with ice for several weeks. In this instance taste and odor did not occur in the raw water such as required treatment until the break-up of the river ice in February, and then a dosage of 1.6 parts per million of Nuchar sufficiently eliminated the taste-producing substances from the plant effluent so that nothing unusual was observed by the consumers.

The above mentioned flow of taste-bearing water coincided in point of time with the annual occurrence of a taste period coming with the

*Presented before the Missouri Valley Section, October, 1936.

initial spring rise of the river in February and sometimes lasting until the so-called June rise (which does not always arrive in June). This constitutes the second of the seasonal periods of taste and odor that have been mentioned, originating outside of the plant. At this time the water has a distinct to strong earthy taste. The water is cold and coagulated with more difficulty than at any other season. The filter operators have a word for it—stubborn—and it is doubtful that a more appropriate and descriptive adjective could be found in the dictionary.

Finally, there is the third seasonal taste and odor period which occurs in the fall, caused by turn-over in the deep sedimentation basins coincident with the first drop in temperature of the basin influent water. This is of course due to the appearance in the basin effluent of a relatively large volume of water which has been for some time in contact with fermenting sludge. To anticipate this period with activated carbon treatment before trouble occurs it is only necessary to observe water temperatures. When the temperature of the influent water first falls below that of the basin water in the lower strata activated carbon treatment at the rate of one part per million is immediately begun.

In all of these cases carbon is applied by means of an Omega dry-feed-machine at the inlet to the fifth and final settling basin, affording about 35 minutes contact before filtration. This has proved to be an effective point of application, and advantageous from the standpoint of economy. Were the carbon introduced earlier, in any of the first four basins for the purpose of securing sludge stabilization in them, the treatment would have to be continuous, whereas by the procedure which has been followed, namely, approximately one-half hour previous to filtration, it has been necessary only a third of the time. A larger dosage also would probably be necessary were it applied before or with the coagulant to become presently incorporated with floc rather than to be permitted to remain in highly dispersed suspension in the water until it reaches the filter beds. One part per million is usually the minimum effective dosage. It has been observed that there is no appreciable effect upon filter runs unless this dosage is almost doubled.

TASTE AND ODOR PROBLEMS AT ST. JOSEPH, MO.*

By John P. Smouse

(Chemist, St. Joseph Water Co.)

A number of articles have been written in the past few years about taste and odor problems at various plants. To add our experiences to those already written might at first seem out of place, but conditions in the water works field are so variable as to make each operators problems differ in some way from those of nearly every other operator. Consequently, our method of treatment may be of value to someone whose case has not as yet been paralleled by conditions described in previous articles. It must be borne in mind that a result obtained with a certain treatment may or may not be obtained at another plant with the same treatment. Hence, evidence from more than one experience must be presented before we can be sure of the results that will be obtained with a certain treatment. Bearing this in mind the experiences with respect to tastes and odors at St. Joseph will be presented.

A brief description of the plant will help clarify the following discussion. The source of supply is the Missouri River. The treatment consists of screening, pre-sedimentation, coagulation, filtration, and sterilization. (See Figure 1.) The screening is accomplished by means of a rotating chain screen. Pre-sedimentation is effected in the No. 1 basin, which has a retention period of 15 hours. The water is coagulated in the No. 2 and 3 basins which are operated in series. The two coagulation basins have a combined retention period of 20 hours. The coagulants which are alum, or alum and lime, are applied as the water flows from No. 1 to No. 2 basin. The chemicals are mixed with the water by a hydraulic jump followed by a slower mix to enable the floc to form. After flowing through the coagulation basins, the water then passes through the rapid sand filters, and is sterilized with chlorine and ammonia when the water temperature is above forty-five degrees. Chlorine only is used when it is below this point. The water is then pumped to the main reservoirs, and flows

*Presented before the Missouri Valley Section, October, 1936.

by gravity to the city. The reservoirs are three in number, are open type, and have a retention period of twenty-four hours. King Hill Reservoir, which is located seven miles across the city, floats on the system. This reservoir is also open type.

Perhaps two of the most common tastes and odors are those produced by the improper control of chlorine dosage, and what might be called the chlorine substitution tastes in water that is free of taste before treatment with chlorine. The routine treatment at St. Joseph

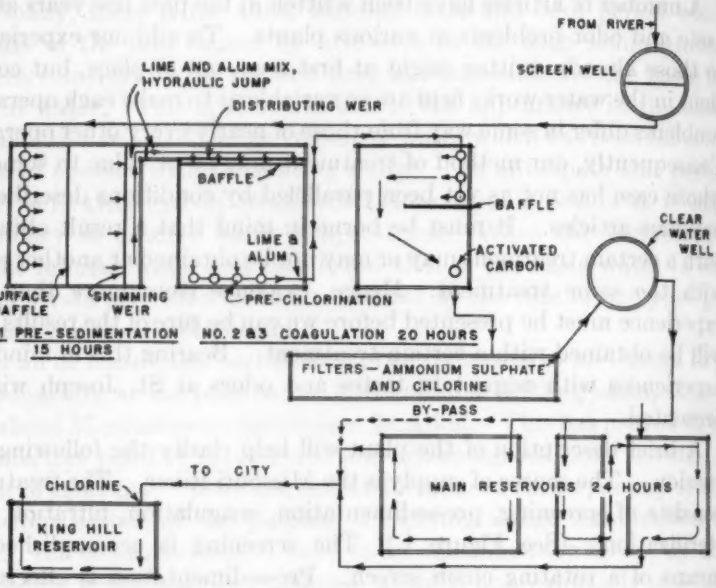


FIG. 1. PROGRESS OF WATER THROUGH THE PLANT AND POINTS OF APPLICATION OF CHEMICALS

is a natural defence against these two offenders. Ammonia is used during the warmer months. During the past five years the ratio of ammonia to chlorine has been varied between the extremes of 1:16 and 1:2. It has been found that under most conditions a ratio of 1:7 gives best results, and this is the ratio now used. This amount of ammonia is theoretically not sufficient to combine with all of the chlorine. The reason for this ratio is to form chloramines only with that portion of the chlorine which does not act on the organic matter in the water in a short period of time. This practice gives a good sterilizing action at the plant, and at the same time carries a sub-

stantial residual through the reservoirs. It might seem that this would not keep down the chlorinous tastes and odors, but the results in this respect have proven to be as good with this ratio as with any other used. There have been no tastes or odors in St. Joseph in the past five years which ammonia in any reasonable quantity would have prevented. A factor which eliminates the taste and odor due to chlorine itself is the twenty-four hour retention period in the reservoirs before the water goes to the consumer. This is sufficient time, during the warmer months, for any active chlorine not in the form of chloramines to leave the water before it leaves the reservoirs. These conditions, combined with daily taste and odor determinations in the laboratory, have eliminated the chlorine and chlorine substitution tastes and odors with one exception, which will be discussed later. During the colder weather chlorine alone is used, but the dosages are reduced sufficiently to carry only a .10 to .15 p.p.m. residual leaving the reservoirs, which has not in the past produced enough taste to be objectionable in the mains.

Since the author came to this plant in June 1931, there have been two occasions when tastes and odors were not controlled by routine treatment. The first occurred in October of 1934, and was a result of algae growing in the reservoirs. The algae during the summer of 1934 were more prolific for two reasons. One was because of the hot dry summer with the high percent of sunshine. The other was the method of treatment used to inhibit the algae. The copper sulphate during this period was applied at the plant, and was not as effective as it has been other years when applied at the reservoirs. As a result of these conditions a complaint was received on the third of October about a taste and odor in the water. Investigation showed this consumer to be located on one of the large feeder lines coming to the city. A trip to the reservoirs showed that the odor was coming from that source, as the same odor was noticeable in the air around the reservoirs, and no odor was in the water as it left the plant. The reservoirs were immediately drained and cleaned. In the meantime enough water was bypassed around the reservoirs to reduce the odor to a point where it was not noticeable to the consumers. Ten complaints were received during this period.

Two types of algae are predominant in the reservoirs during the summer months. One is Cymbella, and the other is similar to Cosmarium except that it is smaller in size. The growth of these is controlled, in the main reservoirs, by application of copper sulphate.

This is applied to the water by means of a sack towed with a boat, and to the banks by means of a spray.

At King Hill Reservoir, which is located across the city and floats on the system, we find that chlorine is the solution to the problem. The reservoir has a capacity of 3.5 million gallons. We apply a dose of eight pounds of chlorine per day during the summer months. There has been no algae trouble in this reservoir since this treatment has been applied over a period of five years. Prior to this treatment, however, considerable trouble was experienced at this reservoir from algae growths.

The second taste and odor trouble occurred Feb. 25, 1936. It was caused by the heavy snows which covered the ground during the entire winter. A large amount of organic matter was carried into the river when the snow began to thaw. As a result, on the morning of Feb. 25 a strong taste and odor appeared in the plant effluent which can best be described as medicinal. This was accompanied by an odor which closely resembled that of phenols. There was no odor, however, present in the river water. The obvious solution seemed to be the application of ammonia, and this treatment was immediately started. A ratio of ammonia to chlorine of 1:16 was first used. A sample taken thirty minutes later showed no improvement. The ratio was then changed to 1:12, and another sample taken showed no improvement. It became apparent that ammonia was not likely to be the solution to the problem. Nevertheless, nine ratios ranging from 1:16 to 1:1 were tried, but with no success. In the meantime in the laboratory samples of the water were chlorinated, and doses of activated carbon were applied. The doses applied were 1, 1.5, 2, 2.5, 3, 3.5, and 4 p.p.m. The samples were stirred for three minutes in a mixing machine and allowed to stand for two hours. All samples showed some reduction in odor. The sample containing the dose of two p.p.m. showing no odor, and all higher doses showing no odor. These results indicated that pre-chlorination followed by activated carbon should be the solution. With this in mind chlorine was applied as the water passed from No. 2 to No. 3 coagulation basin. A dose of 4.5 pounds per million gallons being applied as compared to a dose of three pounds which had been applied in post chlorination before the taste appeared. Activated carbon was applied to the water as it came onto the filters. A dose of 2.75 p.p.m. was used the first day in order to get the filters thoroughly covered. The next day this was lowered to 2.25 p.p.m. The trouble lasted a total of

four days, and at the end of this time had completely disappeared. Four complaints were received at the time we had this trouble. Thanks to the low temperature of the water which helped to mask the taste, and the activated carbon at hand we were able to avoid a situation which would have been very undesirable to say the least.

In conclusion there are three points I wish to mention.

1. Ammonia is satisfactory as a preventive for many tastes and odors, and as an inhibitor of after growths.

2. Chlorine under some conditions will control algae growths.

3. Activated carbon has proven efficient and economical at our plant in the control of tastes and odors of short duration.

TASTES AND ODORS AS FOUND IN THE KAW RIVER WATER*

By Frank E. Willey

(Superintendent, Water Works, Lawrence, Kansas)

The City of Lawrence, Kansas takes its municipal water supply from the Kaw River, a stream having its source in Eastern Colorado, Western Kansas and Western Nebraska. As it flows along from its source until it passes Lawrence it has numerous occasions to pick up great quantities of pollution in the forms of suspended matter, vegetable growths, color and bacteria. The intensity of these various forms of pollution usually varies with the stage of the river and the season of the year. During periods of abnormal rainfall turbidities usually run high and the bacteria count increases to great numbers. Turbidity normally is 1000 p.p.m. ranging up to 25,000 p.p.m. during flood stage. Recently a sample taken at the intake in the river five feet below the surface showed a uurbidity of 280,000 p.p.m. This was a most unusual case and caused myself and the plant superintendent a great deal of anxiety. The stage of the river had raised less than a foot and here we had enough suspended matter in the raw water to fill our settling basins in a few days time. The turbidity of the raw water entering the basins at this time was 45,000 p.p.m. These high turbidities lasted only a couple of days and then began to recede. Odors and tastes usually accompany low water stage and high water stage. During high water stage tastes and odors in the raw water may be described as earthy or musty. They are probably picked up by runoff water entering the stream after it has come in contact with decaying vegetable matter in the soil on the watershed. Activated carbon usually eliminates these odors and tastes satisfactorily if sufficient quantities of this material are used. Algae cause more trouble and are ever present, especially in low river stage during the hot summer months and in the spring of the year, when the ice breaks up or the first freshet comes to cause a turnover in the river bed. From this source, intense odors and tastes develop that have not been eliminated satisfactorily even though dosages of 30 p.p.m. of activated carbon were applied. Split carbon treat-

*Presented before the Missouri Valley Section, October, 1936.

ment was also tried with little or no success. The most recent severe odor and taste epidemic occurred last spring when the ice break came after the prolonged cold weather had frozen the river over solid for several weeks. This was accompanied by high turbidities and a rise in the stage of the river. The treatment applied to the water was alum first and then lime until a hydroxyl alkalinity was built up to approximately 30 p.p.m. (which coagulated the precipitate and suspended matter to give a clear effluent). Then ammonium sulphate, carbon dioxide gas, activated carbon and chlorine were applied. The water then passed through a coagulation basin with a retention of 17 hours. Then alum, activated carbon and carbon dioxide were applied before the final filters and last chlorine. This did not satisfactorily eliminate the odors and tastes from the water. I have learned since that it is a poor practice to apply activated carbon and chlorine at the same stage in the treatment.[1] Later the carbon was applied in the reacting basin ahead of the filters and this resulted in inky water passing the filters when a heavy dose was applied. For the past few months we have been applying the carbon in the coagulating basin with the lime treatment and have had good results even though we have experienced high turbidities since we have been following this practice.

As to phenol tastes in the tap water, we have experienced no trouble in this respect. The river water carries large quantities of phenol wastes and sewage pollution. This is almost constant because of the number of cities discharging raw and treated sewage into the stream above us. The ammonium sulphate treatment in series with the chlorine eliminates most of this trouble. Equal amounts of ammonium sulphate and chlorine are fed and there is maintained a residual chlorine content of 0.15 p.p.m. in the filter influent and 0.40 p.p.m. in the plant effluent. This gives a residual in the outlying districts of town of about 0.10 p.p.m. This amount of chlorine takes care of the bacteria load satisfactorily, rendering a tap water negative for the colon-aerogenes group and with a count of approximately 25 per cc.

No attempt has been made in the past to identify different algae or living organisms in the river water and connect them with the trouble arising from odor and tastes. However at the present time a graduate student in Kansas University is doing research work on these problems. This work has not progressed far enough to have information as to the most prevalent species of algae.

[1] *Editor's Note:* This depends upon the problem to be met.

COMMITTEE REPORT

TENTATIVE SPECIFICATIONS FOR GATE VALVES FOR ORDINARY WATER WORKS SERVICE*

TABLE OF CONTENTS

	SECTION
Type of valve and character of service	1
Supplementary details to be specified	2
Data to be furnished by the manufacturer	3
Size of waterway	4
Basis of structural design	5
Iron castings	6
Bronze	7
Workmanship	8
Bell ends	9
Flanged ends	10
Bodies and bonnets	11
Gates	12
Gate and body rings	13
Wedging device	14
Guides	15
Rollers and tracks for horizontal valves	16
Stem and stem nuts	17
Thrust bearings	18
Stuffing boxes	19
Wrench-nuts	20
Cast iron gears	21
Encased steel gears	22
Indicators	23
By-passes	24
Gaskets	25
Bolts and nuts	26

* Proposed to supersede the "Specifications for Valves" adopted by the American Water Works Association June 24, 1913. Submitted to the Water Works Practice Committee for Committee No. 7 by G. Gale Dixon, Chairman (September 20, 1934). Revised as of December 10, 1935. Second Revision as of December 23, 1936. Third Revision as of January 22, 1937. Comments should be addressed to Harry E. Jordan, Secretary A. W. W. A., 29 West 39th St., New York. *There will be an open discussion of this report at the Buffalo convention on Monday morning, June 7th, 1937.*

SECTION
Cast markings 27
Painting 28
Testing 29
Inspection 30
Preparing for shipment 31

Footnotes:

A.—Relating to Sect. 11.2—Bodies and Bonnets.

B.—Relating to Sect. 13.5—Gate and Body Rings.

Section 1—Type of Valve and Character of Service

1.1 These specifications embrace hand-operated, inside-screw, iron body, bronze mounted gate valves of both the "solid wedge" and "double disc" type, ranging in size from 3 inches to 48 inches in diameter, for ordinary water works service in approximately level setting under operating pressures not exceeding 150 lb. per sq. in.

1.2 These specifications make no provision for the requirements of special conditions of installation or operation, such as:

Power drive;
Installation in vertical or steeply inclined pipe lines;
Water of unusually corrosive quality;
Very frequent operation—as in filter service;
Service in throttled position;
Uniform very low pressure;
Excessive water hammer.

1.3 Such conditions, as well as operating pressures and types and sizes of valves which are outside of the stated limits, are beyond the intended scope of these specifications and will in each case require special consideration in matters of design and construction.

Section 2—Supplementary Details to be Specified

2.1 In purchasing valves under these specifications, it will be necessary to make supplementary specific statement as to the requirements concerning the following details:

2.11 Type of gate valve desired—"solid wedge" or "double disc."

2.12 Type of body ends desired; if bells or flanges of dimensions other than those specified in Sects. 9 and 10, or if other special type of body ends are desired, give substitute specification in full;

2.13 Special uses of bronze, as may be desired;

2.14 Direction in which the wrench-nut shall turn to open

valve—whether right (clockwise) or left (counter-clockwise);

2.15 Wrench-nut—If dimensions are required different from those specified in Sect. 20, state them in full detail;

2.16 Geared Valves—Call for valves to be provided with gears as desired, and state whether horizontal stem with bevel gears or vertical stem with spur gears is required;

(Without further explicit statement, cast iron gears will be furnished, as specified in Sect. 21.)

Call specifically for the following features, if they are desired:

Encased Steel Gears (See Sect. 22);

Indicator (See Sect. 23);

2.17 By-passes—For valves 16 inches in diameter and larger, state whether by-passes are required or not (See Sect. 24);

2.18 Special markings—If purchaser's initials or other special markings are to be cast on the body or bonnet, so state (See Sect. 27);

2.19 Specify drawings and statement of weights, if required (See Sect. 3).

Section 3—Data to be Furnished by the Manufacturer

3.1 The manufacturer shall furnish catalog data, including illustrations and a schedule of parts and the materials of which they are made, in sufficient detail to serve as a guide in the assembly and taking down of the valve and in ordering repair parts.

3.2 When required by supplementary specifications, the manufacturer shall furnish a statement giving the total net assembled weight for each size of valve; and when required shall furnish drawings of adequate size showing the principal dimensions, kind of material and finish of all parts.

Section 4—Size of Waterway

4.1 With the gate open, each valve shall afford an unobstructed waterway of diameter not less than the full nominal diameter of the valve.

Section 5—Basis of Structural Design

5.1 All parts of all gate valves shall be designed to withstand safely and without permanent distortion or injury to any part,

both the stresses resulting from an internal test pressure of 300 lb. per sq. in., and the combined stresses resulting from the full internal service pressure of 150 lb. per sq. in. coincident with the moving of the valve gate across its seat under full unbalanced internal service pressure.

5.2 Further, the valve case and the internal parts of all valves shall be so constructed as to develop the full strength of the valve stem to the point of failure, in operation in either direction under full service pressure, without rupture or permanent deformation of any other part.

Section 6—Iron Castings

6.1 The body, bonnet, gates, and other parts as specified shall be cast from a superior quality of iron remelted in a cupola, or in an air or electric furnace, producing a metal which is tough, of uniform fine grain, without chilled surfaces, readily machined and conforming with the standard specifications of the American Society for Testing Materials, Serial Designation A 126–30, or revisions thereof, for Class B Higher-strength Gray Iron Castings for Valves, Flanges, and Pipe Fittings. All castings shall be clean and sound, without defects of any kind; no plugging, welding or repairing of defects will be allowed.

6.2 From each heat of metal, the manufacturer shall make and test without charge, two deflection test bars, and two tension test bars.

Section 7—Bronze

7.1 All gate valve parts requiring special ability to resist corrosion or requiring smooth surfaces in order to minimize friction, shall be made of bronze or bushed or faced with bronze. This includes stems and stem nuts, thrust bearings, packing glands, gear spindles, gate and body rings, wedging devices, guides, rollers and tracks, and indicator mechanisms, all as hereinafter specified in detail.

7.2 All bronze parts shall be made of metal of known and approved composition.

7.3 Valve stems and stem collars and stem nuts shall be cast or forged from billets of manganese bronze or other suitable approved bronze alloy having a tensile strength of not less than 60,000 lb. per sq. in., a yield point of not less than 30,000 lb. per sq. in. and an elongation of not less than 15 per cent in 2 inches.

7.4 Bronze alloy for peened or rolled-in gate rings shall be sufficiently malleable to conform to dovetailed grooves and shall have a minimum compressive strength, without deformation, of 4,000 lb. per sq. in.

7.5 Bronze alloy for other valve parts shall have a tensile strength of not less than 30,000 lb. per sq. in., a yield point of not less than 15,000 lb. per sq. in. and an elongation of not less than 15 per cent in 2 inches.

7.6 The manufacturer shall make and test not less than one test bar from each heat of metal, in accordance with the American Society for Testing Materials specification for the material used.

7.7 All bronze facings and bushings shall be of ample dimensions.

Section 8—Workmanship

8.1 All foundry and machine work shall be done in accordance with the best modern practice for the class of work involved.

8.2 All parts shall conform accurately to the required dimensions and shall be free from injurious defects. All machined parts shall be made to template or gage.

8.3 All joints shall be faced true and shall be water-tight where subject to water pressure.

8.4 The backs of the end-flanges of all flanged end valves 16 inches in diameter and larger shall be spot-faced around the bolt holes to uniform thickness.

8.5 All iron parts receiving bronze mounting shall be made true and smooth and the bronze mounting shall be finished to fit.

8.6 Such hand work shall be done in finishing as is required to produce a neat, workmanlike, well-fitting, and smoothly operating job throughout.

8.7 All parts of valves of the same size and same make shall be interchangeable.

Section 9—Bell Ends

9.1 The dimensions of the sockets of bell-end gate valves shall conform to the dimensions for "Class C" special castings as required in the American Water Works Association Standard Specifications for Cast Iron Pipe and Special Castings, unless explicitly otherwise provided in supplementary specifications.

9.2 The thicknesses and disposition of metal in the bells shall be such as to produce strength not less than that of bells of "Class D" dimensions under the same Standard Specifications.

Section 10—Flanged Ends

10.1 The end-flanges of flanged end gate valves shall conform in dimensions and drilling to the "American 125 lb. Cast Iron Flange Standard" as promulgated by the American Standards Association, unless explicitly otherwise provided in the supplementary specifications.

Section 11—Bodies and Bonnets

11.1 Valve bodies and bonnets shall be made of cast iron, ruggedly constructed for resistance to distortion under all operating and test conditions. (See foot-note "A".)

11.2 It is recognized that some variations are unavoidable in the making of patterns and castings. In order to define what may be considered a reasonable degree of accuracy in shell thickness measurements, the following inspection limits shall be allowed:—shell thickness measurements taken at points diametrically opposite shall, when added together and divided by two, equal or exceed the minimum metal thicknesses given in the drawings. Shell thicknesses at no point shall be more than 12½ per cent below the minimum metal thicknesses called for in the drawings, and no continuous area of deficient thickness shall exceed 12½ per cent of the superficial area of the casting.

11.3 The necks of all flanges shall be thickened, ribbing shall be properly proportioned, and corners shall be coved and rounded, in accordance with the best foundry practice.

11.4 Threads in bodies shall be accurately cut and of ample depth to secure the seat rings, and the seats shall be machined to fit the backs of the rings.

11.5 The thrust bearing recess and the stem opening in the bonnet shall be finished. In valves 16 inches in diameter and larger, the thrust bearing recess shall be bronze lined, and the stem opening shall be bronze bushed.

11.6 In horizontal stem valves 16 inches in diameter and larger, and in vertical stem valves 24 inches in diameter and larger, a pair of accurately matched dowel holes and tapered or round-end dowels shall be provided in the flanges of body and bonnet to aid in assembling. Dowels shall be located at opposite ends of the flange, one on the lateral center line and the other about two inches therefrom.

Section 12—Gates

12.1 Gates may be of the "solid wedge," "double disc parallel seat," or "double disc inclined seat" type.

12.2 In valves 4 inches in diameter and smaller, the entire gate assembly shall be made of solid bronze. In larger valves, the gates shall be made of cast iron unless bronze is explicitly required by supplementary specifications.

12.3 Gates shall be ruggedly constructed for resistance to deflection.

12.4 Cast iron gates shall be accurately machined to receive the bronze gate rings. Where gate rings are to be rolled or peened in, the grooves receiving the ring metal shall be machined.

Section 13—Gate and Body Rings

13.1 Gate and body rings shall be made of cast bronze, using metal in the gate rings materially softer than in the body rings, with cross-section sufficiently stiff to resist accidental deformation in handling and assembly.

13.2 Body rings shall be back-faced, the threads accurately cut, and the rings screwed into machined seats in the body.

13.3 Gate rings shall be rolled, peened or pressed into machined grooves in the faces machined in the gates.

13.4 Finish cuts shall be taken over the rings after they are finally secured in place.

13.5 The width of face of seat rings shall be appropriate to the hardnesses of the metals used in gate and body rings and to the size and general design of the valve. (See foot-note "B.")

13.6 The width of face of the gate rings shall be not less than that of the seat rings, and such greater width shall be provided as is practically required in order that the gates may continue to seat tightly after allowing for reasonable wear of the faces of the rings and of the various parts of the mechanism of the gate.

Section 14—Wedging Device

14.1 Gate valves of the "double disc parallel seat" type shall be equipped with an internal wedging device which, in closing the valve will operate, when the gates reach position opposite the ports, to spread the gates apart and press them firmly against the seats. The mechanism shall be so constructed that it will release with the first half-turn of the stem in starting to open the valve.

14.2 The wedging device shall be simple and rugged in design and shall operate freely and positively with the valve in normal position, both in wedging the gates into contact with the seats and in releasing them therefrom.

14.3 In valves 4 inches in diameter and smaller, the entire wedging mechanism between the discs shall be made of solid bronze.

14.4 In valves 16 inches in diameter and larger, all wedging surfaces shall be bronze to bronze; in valves of diameter from 6 inches to 12 inches, inclusive, wedging surfaces may be bronze to iron.

14.5 Pins and bolts in the wedging mechanisms of all valves shall be made of bronze.

Section 15—Guides

15.1 In solid wedge gate valves, tongue-and-groove guides shall be provided on the sides of the gate and in the case, to keep the gate centered between the seats throughout its full length of travel. In valves 16 inches in diameter and larger, guide contacts shall be bronze to bronze.

15.2 In double disc gate valves, the case shall be filleted to reduce the side clearance of the gates.

Section 16—Rollers and Tracks for Horizontal Valves

16.1 Gate valves 16 inches in diameter and larger, designed to lie on their sides, with horizontal stems, shall be equipped with solid bronze or hard babbitt tracks securely fastened in body and bonnet, carrying the weight of the gates, throughout their entire length of travel, on rollers or shoes.

16.2 In solid wedge gate valves, the tongue-and-groove gate guides specified in Sect. 15 may be modified to serve this purpose also, using solid bronze rollers instead of bronze shoes to carry the load in valves 20 inches in diameter and larger.

16.3 In double disc gate valves of the "rolling disc" type, the discs will serve as rollers, operating on bronze or hard babbitt tracks.

16.4 In double disc gate valves of other design, the discs shall be carried on solid bronze rollers securely attached to them, except that in valves 16 inches in diameter bronze shoes may be used instead of rollers.

16.5 In all valves in which rollers are used, bronze scrapers shall be provided to traverse the tracks ahead of the rollers in both

directions of travel, removing any foreign matter which may have accumulated on the track.

Section 17—Stems and Stem Nuts

17.1 Valve stems and stem nuts shall be cast or forged or rolled and forged from billets of manganese bronze, or other suitable noncorrodible metal having physical qualities which meet the requirements specified in Sect. 7. Stem collars shall be cast or forged solid with the stems.

17.2 The threads of stems and stem nuts shall be of the square, Acme, modified Acme or ½ V type, cut with a sufficient number of cuts to avoid straining the metal.

17.3 Stems shall be turned and threaded straight and true, and shall work true and smooth and in perfect line throughout the lift of opening and thrust of closing the valve.

17.4 The diameters of stems at base of thread and number of threads per inch for determining pitch shall be not less than the following:—

Diameter of Valve, Inches	*Minimum Diameter of Stem at Base of Thread, Inches*	*Minimum Number of Threads per Inch*
3	0.8594	3
4	0.8594	3
6	1 1.00	3
8	1 1.00	3
10	1⅛ 1.125	3
12	1¼ 1.25	3
16	1½ 1.50	3
20	1¾ 1.75	3
24	2 2.00	2
30	2½ 2.50	2
36	3 3.00	2
42	3½ 3.50	2
48	4 4.00	2

17.5 As an alternate to the stem sizes specified in the preceding paragraph for valves 30 inches in diameter and larger, stems of diameters not less than those shown in the following tabulation will be accepted when made from metal having a tensile strength of not less than 80,000 lb. per sq. in., a yield point of not less than 40,000 lb. per sq. in., and otherwise meeting the requirements of these specifications:

Diameter of Valve, Inches		*Minimum Diameter of Stem at Base of Thread, Inches*
30	$2\frac{3}{8}$	2.375
36	$2\frac{3}{4}$	2.75
42	3	3.00
48	$3\frac{1}{2}$	3.50

Section 18—Thrust Bearings

18.1 Thrust bearings shall be designed to develop safely the full strength of the valve stem. The housing for the thrust bearing shall be carefully machined, and in valves 16 inches in diameter and larger, shall be bronze lined.

Section 19—Stuffing Boxes

19.1 Stuffing boxes shall be made of cast iron, with stem opening, thrust bearing recess and bonnet face carefully machined. In valves 16 inches in diameter and larger, the stem opening shall be bronze bushed, and the thrust bearing recess shall be bronze lined.

19.2 Stuffing boxes shall have depth not less than the diameter of valve stem to be passed, and internal diameter adequate to receive sufficient packing to insure no leakage.

19.3 Glands for valves 12 inches in diameter and smaller shall be made of solid bronze; for larger valves, glands may be made of cast iron with bronze bushings. The bottoms of glands shall be concavely beveled to an angle of not less than 15 degrees from a plane normal to the axis of the valve stem.

19.4 Gland followers, where used, may be made of cast iron.

19.5 Gland bolts and nuts shall be made of bronze of the same quality as the stem metal; bolts shall be rolled and forged or cast.

19.6 Stuffing boxes shall be packed with best quality soft graphited hydraulic packing, made of flax of medium fiber length or of properly lubricated braided asbestos.

Section 20—Wrench-Nuts

20.1 Wrench-nuts shall be made of cast iron, fitted to the top of the valve stem and secured by nut or key.

20.2 Except as explicitly otherwise provided in supplementary specifications, the wrench-nuts shall be $1\frac{15}{16}$ inches square at the top, 2 inches square at the base, and $1\frac{3}{4}$ inches high, with a flanged base upon which shall be cast an arrow at least 2 inches long showing the direction of opening and the word "OPEN" in distinct $\frac{1}{2}$ inch letters.

Section 21—Cast Iron Gears

21.1 Except as otherwise explicitly required by supplementary specifications, geared valves shall be equipped with gears made of cast iron of the quality specified in Sect. 6, accurately formed and smooth running, with bronze spindles running in bronze or babbitt lined bearings of ample size.

21.2 Gearing shall be designed for maximum facility and speed of operation with a minimum number of men.

Section 22—Encased Steel Gears

22.1 When steel gears are explicitly called for by supplementary specifications, geared valves shall be equipped with machine cut cast steel gears, accurately formed and smoothly finished, with bronze spindles running in bronze or babbitt lined bearings of ample size enclosed in oil-tight cast iron gear case mounted on brackets attached to the bonnet of the valve in such manner as to permit repacking the stuffing box of the valve without disassembly.

22.2 Steel gearing shall be designed for maximum facility and speed of operation with a minimum number of men, and gear ratios shall be not less than the following:

Diameter of Valve, Inches	*Gear Ratio*
16	2:1
20	2:1
24	2:1
30	3:1
36	3:1
42	4:1
48	4:1

22.3 Steel castings shall meet the requirements of the Standard Specifications of the American Society for Testing Materials, Serial Designation A-27-24 for carbon steel castings, Class B, soft, except that the sulphur content shall not exceed 0.05 per cent.

Section 23—Indicators

23.1 When required by supplementary specifications, geared valves shall be equipped with indicators to show the position of the gates in relation to the ports. The indicator mechanism shall be made of bronze or other non-corrodible metal throughout, mounted in a heavy dirt-proof cast iron case.

Section 24—By-passes

24.1 By-passes when required shall be of the following sizes:

Diameter of Valve, Inches	*Diameter of By-pass, Inches*
16–20	3
24–30	4
36–42	6
48	8

Section 25—Gaskets

25.1 Water-tight flanged joints shall be made up with paper gaskets, full cut with holes to pass bolts. Gasket material shall be free from corrosive alkali or acid ingredients.

Section 26—Bolts and Nuts

26.1 All bolts and nuts shall be American Standard, and, except as otherwise specified, shall be made of steel conforming with the Standard Specifications of the American Society for Testing Materials, Serial Designation A 107-33 for medium-carbon open hearth hot rolled bar steel, and shall be rust-proofed by hot dip galvanizing, Parkerizing, Sherardizing, or other approved process.

26.2 Bolts and nuts shall be "semi-finished" when bearing is on finished surfaces.

Section 27—Cast Markings

27.1 Each valve shall be plainly marked with the manufacturer's name or particular mark, and with the year it was made, the size of the valve, and the designation "150 W." indicating the working water pressure, all cast on the bonnet or body.

27.2 When required by supplementary specifications, special markings to the number of not more than four additional letters or figures shall also be cast thereon.

Section 28—Painting

28.1 All ferrous parts of the valves, except finished or bearing surfaces, shall be given two coats of asphaltum varnish or pipe dip. After the valves are assembled and tested a third coat shall be applied on the exterior.

Section 29—Testing

29.1. After completion, each gate valve shall be tested at the shop for performance in operation, and for water-tightness and resistance to distortion under internal pressure.

29.2 Each valve shall be operated, in the position which it will assume in service and for the full length of gate travel in both directions, to demonstrate the free and perfect functioning of all parts in the intended manner; any defects of workmanship shall be corrected and the test repeated until satisfactory performance is demonstrated.

29.3 Each valve shall be subjected to hydrostatic tests under pressures of both 300 and 100 lb. per sq. in. The tests shall be conducted in the following manner:

29.31 For solid wedge gate valves—the hydrostatic pressure shall be applied through bulkheads alternately to the two sides of the closed gate with the opposite side open to inspection, and again with both ends bulkheaded and the valve open.

29.32 For double disc gate valves—the hydrostatic pressure shall be applied inside the bonnet with gates closed.

29.4 Under the 300 lb. test there shall be no leakage through the metal or permanent deformation of any casting. Under the 100 lb. test all joints and seats shall be perfectly water-tight.

Section 30—Inspection

30.1 All work under these specifications shall be subject to inspection and approval by the purchaser's duly authorized engineer or inspector, who shall at all times have access to all places of manufacture where materials are being produced or fabricated or tests are being conducted, and shall be accorded full facilities for inspection and the observation of tests. Any gate valve or part which he may condemn as not conforming to the requirements of these specifications shall be made satisfactory or shall be rejected and replaced.

Section 31—Preparing for Shipment

31.1 Gate valves shall be complete when shipped, and the manufacturer shall use all due and customary care in preparing them for shipment so as to avoid damage in handling or in transit. Particu-

lar care shall be taken to see that all valves are completely closed before shipment. Valves 24 inches in diameter and larger shall be securely bolted or fastened to skids in such manner that they may be safely unloaded with a crane.

The above specifications are submitted for study and discussion. They will be submitted to the Board of Directors for approval at the meeting in Buffalo during the week of June 7, 1937. If then approved, and if no modifications are thereafter made, they will be formally promulgated at the time of the annual meeting of the Board in January, 1938. Until such formal promulgation is made, the specifications are in no wise official.

For the Committee on Transmission and Distribution
G. GALE DIXON, *Chairman*
For Sub Committee 7-E on Valves
WILLIAM R. CONRAD (Ch.)
HAROLD W. GRISWALD
FRANK H. STEPHENSON
WILLIAM FLANNERY
J. ARTHUR JENSEN

* * * * *

FOOTNOTES

Owing to the disturbance of some manufacturing standards which would result, the Committee refrains from incorporating the requirements (A and B below) in the specifications at this time.

A—Relating to Sect. 11.2—Bodies and Bonnets

It is the Committee's judgment, based on a study of practice, that in order to insure adequate stiffness of cast iron valve bodies, bonnets and gates in resistance to distortion under internal pressure, these parts should be made with minimum thicknesses of shell approximating those of "Class D" standard cast iron pipe of corresponding diameter.

In deference to the expressed wish of the Manufacturers, such a provision has not been included in the specifications.

The following tabulation records the Committee's recommendation on this point:

Diameter of Valve, Inches	Minimum Thickness of Shell, Inches	Diameter of Valve, Inches	Minimum Thickness of Shell, Inches
3	$\frac{1}{2}$	20	1
4	$\frac{1}{2}$	24	$1\frac{1}{8}$
6	$\frac{9}{16}$	30	$1\frac{3}{8}$
8	$\frac{5}{8}$	36	$1\frac{9}{16}$
10	$\frac{11}{16}$	42	$1\frac{13}{16}$
12	$\frac{3}{4}$	48	2
16	$\frac{7}{8}$		

B.—Relating to Sect. 13.5—Gate and Body Rings

Quite wide variation exists in current practice regarding the width of seat rings, especially in the smaller sizes of valves. The optimum width is obviously dependent upon the hardnesses of the metals used in the rings, and upon the extent to which sliding friction between gate and seat rings is developed in the process of seating and unseating the gate; the latter factor being dependent in turn upon the type of mechanism employed. Again, the smaller valves are generally subject to more frequent operation and hence more rapid wear than are large ones, and a proportionately greater force is more readily applied by man-power in operating them.

A study of practice has led the Committee to conclude that the following tabulation and text provides reasonable and proper minimum dimensions for gate and body rings:

Minimum Finished Dimensions for Gate and Body Rings (Inches)

Diameter of Valve	Body Rings			Gate Rings
	Width of Face	Thickness of Flange	Length of Threaded Neck	Minimum Thickness
3	$\frac{3}{8}$	$\frac{5}{32}$	$\frac{3}{4}$	$\frac{5}{32}$
4	$\frac{3}{8}$	$\frac{5}{32}$	$\frac{3}{4}$	$\frac{5}{32}$
6	$\frac{7}{16}$	$\frac{3}{16}$	$\frac{7}{16}$	$\frac{5}{32}$
8	$\frac{1}{2}$	$\frac{3}{16}$	$\frac{7}{16}$	$\frac{5}{32}$
10	$\frac{9}{16}$	$\frac{3}{16}$	$\frac{1}{2}$	$\frac{3}{16}$
12	$\frac{5}{8}$	$\frac{3}{16}$	$\frac{1}{2}$	$\frac{3}{16}$
16	$\frac{3}{4}$	$\frac{7}{32}$	$\frac{5}{8}$	$\frac{7}{32}$
20	$\frac{7}{8}$	$\frac{1}{4}$	$\frac{11}{16}$	$\frac{1}{4}$
24	1	$\frac{5}{16}$	$\frac{3}{4}$	$\frac{1}{4}$
30	$1\frac{1}{4}$	$\frac{11}{32}$	$\frac{7}{8}$	$\frac{9}{32}$
36	$1\frac{1}{2}$	$\frac{15}{32}$	1	$\frac{5}{16}$
42	$1\frac{3}{4}$	$\frac{9}{16}$	$1\frac{1}{8}$	$\frac{11}{32}$
48	2	$\frac{3}{8}$	$1\frac{1}{4}$	$\frac{3}{8}$

The gross thickness of the threaded neck—measured to top of thread—of body rings shall be not less than the thickness of flange. Dovetailing of gate rings shall extend into the cast iron gate below the faced surface matching the back of the ring to a depth equal to the minimum thickness of the ring.

The given widths of body rings result in a bearing pressure of 2,000 lb. per sq. in. under the hydrostatic test pressure of 300 lb. per sq. in., for valves 24 inches in diameter and larger; for valves smaller than 24 inches in diameter, the bearing pressures progressively decrease.

NEW PUBLICATIONS

Public Works Planning[1]

This report, dated December 1936, was prepared by the National Resources Committee. It is divided into three sections. The first contains certain recommendations made by the National Resources Committee. The second contains the recommendations of the Water Resources Committee concerning drainage basin problems and programs. The third section presents an outline of principles and policies which theoretically should govern the allocation of costs of public works.

The second section covers fairly familiar ground for men in the water supply industry in the form of a collection of drainage basin studies and a listing of various projects.

It was prepared under the direction of Abel Wolman, chairman of the Water Resources Committee and also chairman of this Association's Committee on National Water Policy.

"These studies were undertaken early in 1936 at the request of the National Resources Committee, with the following objectives:

(1) To determine the principal water problems in the various drainage areas of the country.
(2) To outline an integrated pattern of water development and control designed to solve those problems, and
(3) To present specific construction projects and investigation projects as elements of the integrated pattern or plan, with priorities of importance and time.

The findings of the investigation are reasonably clear on the first and second items of the assignment. The third item presented special difficulties, particularly since it necessitated review and appraisal by the Committee of thousands of projects, aggregating billions of dollars in estimated cost and relating to water-resource development during a period of some 10 years.

Action on certain highly controversial projects was deferred either

[1] Public Works Planning. A report of the National Resources Committee. U. S. Government Printing Office. $0.60.

because there was not time for an adequate review of the complex and, in some instances, voluminous material involved, or because data essential to sound action were lacking. In the latter case, further investigation was recommended. Completion of certain projects already constructed in large part was recommended even though their original merit appeared to be gravely in doubt; abandonment of them now with little or no salvage seemed wholly impracticable. Investigations under way and construction projects contingent upon a favorable outcome of investigations in progress or now recommended are not listed in most instances."

The report outlines certain characteristics which it believes the national planning activity should have.

"1. It will be concerned, in the final analysis, not with water per se, but with the promotion of public safety, public health, the public convenience and comfort, the economic welfare of the public, the establishment or maintenance of a high standard of living.

2. It will seek to promote the maximum integrated use and control of water, within the shifting limits imposed by considerations of technical feasibility and of economic and social justification.

3. It normally will treat drainage areas as units with respect to their waters in recognition of the train of connected problems that runs from river source to river mouth.

4. Though treating drainage areas as units, it will scrupulously observe the rights of the several States both in intrastate and interstate streams, seeking through coöperation to promote the full use of water in the best ways and in the proper places.

5. It will recognize and abide by the axioms that facts are indispensable prerequisites to sound action with respect to water, and that conclusions and commitments not based on predetermined facts almost certainly prove indefensible. In keeping with this recognition, it will promote by all feasible means continuous assembly of the basic data essential for an evolving, unending water plan. At the risk of repetition, the statement may be made categorically that the adoption of a plan and program for the use and control of the waters of any river basin without a definitive study of adequate data bearing on all phases of the interlocking physical and cultural problems involved would be illogical, would result in economic waste, would invite controversy, and might preclude desirable action later.

6. It will assign the cost of constructing and operating projects undertaken from time to time as suitable elements in an evolving

water plan among the agencies concerned in as close accordance as possible with the distribution of benefits. The proposition that those who will profit from a given undertaking normally should pay, in a manner not involving avoidable or undue hardship, in proportion to the advantage they will reap should be the basic principle of action.

7. In determining whether or not water projects are justifiable, and in distributing the costs of meritorious projects among the beneficiaries, it will take properly into account social benefits as well as economic benefits, general benefits as well as special benefits, potential benefits as well as existing benefits, wherever they are involved. Some of these benefits are not capable of measurement, and accordingly they commonly have been ignored in the past in evaluating certain types of enterprises. They are subject to reasonable appraisal, however, and their intangible nature will not justify their neglect in the future. In great measure, they concern the public at large. A public water policy should assiduously conserve and promote public interests. To this end, social accounting must take its place with economic accounting. As effective water planning proceeds year after year without interruption—planning based on fundamental and exhaustive engineering, economic, and social studies that cover all relevant conditions—there inevitably will result not only a clearer understanding of what constitutes the public interest, but also a greater opportunity to promote it through equitable control and orderly development of water resources."

There are seventeen sectional or district summaries covering the areas from New England to the Pacific Northwest and from the Southeast to California. The various projects listed include water supply, pollution control, drainage, erosion control, navigation, recreation, flood control and water power. They are grouped into three grades:—A—ready for immediate study or construction; B—projects, which, while desirable for immediate study or construction, involve unsolved questions of public policy, etc.; C—projects which have a definite relation to drainage basin improvements, but which appear to merit only deferred handling.

It is noted that the various dam construction projects in the Tennessee Valley are listed under the tripartite heading of "flood control, navigation and water power." It is also noted that the Passamaquoddy project and the Florida ship canal are given an "A" rating for investigation. Throughout the report, in the sections

relating to various drainage basins, one notes projects listed properly in an "A" rating, the need of which has long been manifest to persons familiar with the facts, but upon which no action has been or is in progress, largely because of real or reputed lack of funds in the area concerned. At points where local controversies exist over the merit of one or another project, the report makes no selection. One may inquire whether, in certain instances, the report does not over faithfully reflect the statements made by local authorities and lack the effective imprint of the engineering knowledge and national viewpoint possessed by the committee? In other words, is the report truly one of "public works planning" or is it a collected listing of proposed work for which federal aid has been requested? An inclusive positive rejoinder to that question would not be correct because it is reasonably clear that certain conflicting projects in single drainage areas have been put into proper adjustment.

But it is in the first and final part of the report that the more basically significant material is found. If by some chance the policy of federal participation in public works relating to water resources is abandoned, part two still would remain a valuable listing of water conservation projects. Parts one and three imply the permanence of federal participation in public works upon the pattern developed during the recovery period.

The National Resources Committee recommends: (1) unified plans for many purpose projects; (2) selection and analysis of projects; (3) advance programming of public works; (4) adoption by Congress of a 6-year program; (5) correlation of timing of public activities with other public fiscal policies through a fiscal advisory committee appointed by the President; (6) the lump sum method of appropriation; (7) the correlation of federal activities with regional, state and local public works both for the full value of the works themselves and for the relationship among the projects in construction or operation; and, (8) agreement and adoption by legislative authorities of procedure for division of responsibility and costs.

The committee further recommends that an agency similar to the Public Works Administration be charged with the task of determining order of work, the agencies responsible for each undertaking and the allotment of funds. "Construction by established Federal, State or local agencies can be provided through the negotiating and allotment procedure outlined above."

The third part of the report deals with the division of cost of public

works. It discusses the public works program of the recovery period as a balancing mechanism for the cycles of expanding and contracting activity. It properly points out that it has its value during the lag phase of free enterprise and not when superimposed upon the active period.

It is shown that underlying any program of public works, whether it be that of the normal or the depression period, there are questions concerning the most equitable methods to use in spreading the payment of costs of such work. Shall they be divided on the basis of governmental functions? In proportion to relative benefits? Upon a per capita basis? On the basis of a fixed percentage contribution? On the basis of relative wealth and income? By formula designed to accomplish special purposes?

The conclusion is that no single one of these methods is fully equitable nor can any fixed formula for proportionate application of one or all of the methods be just. "Only through some form of administrative determination can all the elements in a complex economic, social and political situation be carefully weighed." The concluding paragraphs of this section deserve full quotation:—

"The division of costs of public works should be viewed as a problem in the incidence of taxation to defray the cost of capital improvements rather than as a problem in the apportionment of costs among units of government. The fundamental question for determination is what citizens or groups of citizens should be called upon to defray the cost of particular improvements. The answer to this question is difficult because the principles upon which a division can be based are not clear. Nor do past precedents furnish a safe guide for the development of future policies. The divisions have been effected mainly by way of grants, though emergency loans have recently been employed. The formulas which have been utilized have been both simple and arbitrary and have not taken account of cyclical factors, the incidence of benefits, availability of local revenue, or of many other factors. While many factors need to be considered in developing policies in redivisions of cost, the weight to be attached to each and the formulas themselves have to be developed. Even the interest of citizens and particular governments served by public works have to be evaluated."

"The development of improved methods for dividing the costs of public works, therefore, seems to depend upon the prosecution of further research into the subject and an increase in the rôle of

administrative authorities. Congress should define the scope of the public-works program, indicate the extent of the national interests, fix the limits on Federal assistance, and determine the amount of annual and reserve appropriations. The development of specific plans and the division of costs, within the limits set forth, should be determined by a Federal public-works agency working in close cooperation with representatives of subordinate governments. These plans should be developed as a coöperative enterprise rather than as a Federal bounty If undertaken in coöperative fashion divisions of cost may be worked out in conference and voluntarily assumed. The results possible of achievement under such a plan seem to offer greater prospect of success than if burdens are imposed on subordinate governments (even through grants) by the national unit."

"In order to provide for the proper timing of public works, and the adoption of appropriate financial policies therefore, as well as to coordinate these with the fiscal and monetary policies of the Nation, final decisions on these matters should be lodged with a fiscal advisory committee composed of the President, the Secretary of the Treasury, the Director of the Budget, the chairman of the National Resources Committee, and the chairman of the Board of Governors of the Federal Reserve System. Their task would be to approve the physical and financial features of the public-works program, to determine the volume of activities within the limits fixed by Congress, and to time the commencement and cessation of public-works activities."

The implications of this document as a whole can hardly be overestimated. One section has been seen to contain a list of water resource development projects upon a national scale. Every watershed—every region of the continental United States has been scanned and the tabulation includes the project fruit of that scanning in terms of relative merit either for investigation or construction.

(One may assume that national planning is not to be confined to water resources alone. While the report touches these only, one cannot lose sight of parallel interest in land use, transportation and other forms of national activity. The principles laid down for water planning must be interpreted in terms of their applicability to other fields.)

Then the National Resources Committee recommends that in order to obtain orderly progress in execution of these projects, there be created an administrative board. This board would be given

lump sum appropriations and the power to establish priorities and to distribute the funds. Finally, into the hands of an advisory board, headed by the President, would be placed the power to determine the time of beginning and of ending public works activities, plus the power further to determine their total volume and to allocate the costs of such works.

In certain of their features, these proposals are similar to the course recently in effect in the handling of recovery funds. Again there are points of similarity with the proposals of Walter Lippmann in his "Method of Freedom." Advocates of every form of government from Samarkand to Sandusky can find in the proposals things to support and things to condemn.

The handling of the report by the public press may be symptomatic of the attitude of the public concerning it. It was generally noted in terms of a list of projects in the tributary area. Does that imply that the recovery activities are still naïvely thought of in terms of getting Federal funds for local projects? Those who feel that such is the implication of the report should study it carefully. They will find that it goes much further than that. It proposes to make into a permanent part of American institutions many of the methods developed during the past four years and to implement them with appropriations and power to such an extent that all public works activity in the country would in effect be Federally controlled. It is to be hoped that a wide understanding of the report can be developed and that the nation will make its decision intelligently and not at the receiving end of a tin cup.

Harry E. Jordan.

ABSTRACTS OF WATER WORKS LITERATURE

Key: JOURNAL of the American Water Works Association, 29: 1, 10, January, 1937. The figure 29 refers to the volume, 1 to the number of the issue, and 10 to the page of the JOURNAL.

FLOODS—1936

Engineering Reports on Record Floods from New England to the Potomac. Eng. News-Rec., 116: 441–2, March 26, 1936. Flash floods, March 18–21, rising to unprecedented heights, inundated river valleys of northeastern section of country, following heavy rains which fell on ground previously saturated by rain and melting snow. Nearly every large river in 200,000-square mile region from Canadian line to Virginia and Ohio set new records of flood height and peak flow volume, submerging lower districts of valley cities and causing losses that are certain to reach totals of several hundred millions of dollars. Nearly 200 lives were lost. Characteristic of all floods was their sudden rise. Preliminary figures indicate that some cities may have experienced so-called 1000-year flood, or higher. Headwater detention reservoirs proved effective on the relatively few rivers on which they have been constructed. Flood rehabilitation allotment of $43,000,000 has been made by President: WPA is to repair or replace damaged public property. **Johnstown Flood Rivals '89 Dam Break.** H. W. RICHARDSON. 442–4. Flood waters at Johnstown, Pennsylvania, at crest depth of 18 feet above curb at city hall, approached within 1.5 feet of flood following historic dam break in 1889; excepting which, highest previous flood was that recorded as stage 18.6 in 1907; this year's flood reached stage 27. Mud and debris, after flood had subsided, was as much as 8 feet deep. Total damage of $33,000,000 is fully one-third of assessed valuation of whole city. Fortunately, city's water supply, derived from mountain reservoirs, did not fail. **Pittsburgh Experiences Maximum Flood.** NATHAN B. JACOBS. 444–6. Simultaneous arrival at Pittsburgh of flood crests of Allegheny and Monongahela Rivers, which join in city to form Ohio River, resulted in worst flood in history, gage reaching height of approximately 46 feet, more than 7 feet higher than previous record in 1907. Flood commission appointed at latter time predicted that gage height of 43 feet might occur. Flood stage is about 25 feet; this has been experienced more than 80 times in last century. Maximum rate of flow from total drainage area of 2 rivers (18,920 square miles) was 34 second-feet per square mile; but indicated runoff from Monongahela and lower Allegheny areas, where bulk of precipitation occurred, was 45 second-feet. Computed discharge in 1907 flood was 24.3 second-feet. Water is supplied in Pittsburgh by 3 privately owned plants in addition to municipal works. Difficulties experienced are outlined briefly. Ross Station, city low-lift pumping station, was out of commission from evening of March 17 to noon on March 21. City was entirely

without power or light for part of time. **Susquehanna Floods Highest on Record.** H. W. RICHARDSON. 446 and 466. Flood crests on Susquehanna River and its main tributaries, the Juanita and West Branch, synchronized and previous high-water records of June, 1889, were exceeded by from 2 to 5 feet. At Harrisburg, crest reached stage of 30.4 feet, 3.3 feet higher than record. Water supplies of most of valley towns are derived from storage reservoirs in mountains and were largely unaffected. At 10 p.m. on 18th in Harrisburg, which draws its supply from river, pumping station was disabled and on afternoon of 19th, water overtopped dikes around filter plant which were supposedly higher than possible flood stage, putting plant out of commission. Reservoirs, which have capacity of 53 million gallons, contained 35 million gallons, about 4 days' supply. To conserve water, all factories were ordered to discontinue operation and a private water company aided by pumping 1 m.g.d. into system. No important dam in whole state failed. **Hydrology of New England Floods.** H. K. BARROWS. 466–7. Temperature, precipitation, snow depth, and runoff data are given for greatest and most widespread floods ever known in New England, indicating that basic cause was heavy rainfall and temperature sufficiently high to melt snow over wide area. It is estimated that, for peak discharge at Vernon on Connecticut River, total runoff during flood of March 18–22 would be about 4.5 inches over drainage area of 6,220 square miles. Probably upwards of 3.5 inches had already passed off during previous week, so that some 8 inches of runoff occurred in period March 12–22. Effect of reservoir control was again strikingly demonstrated by conditions on Deerfield River, on which flood waters were retained in reservoirs, and indicates how such floods may be materially lessened. **New England Flood Exceeds That of 1927.** V. T. BOUGHTON. 467–8. Flood flows exceeded those of 1927 with one exception, Winooski River in Vermont, where 2 flood control reservoirs have been erected since 1927 disaster. On March 18th, old Greenwood Dam at New Hartford on Farmington River failed. Dam was old timber structure, built 90 years ago, about 150 feet long and 20 feet high. Examination of small remaining section indicated that buried timbers were badly decayed. Fortunately, failure occurred before flood crest reached lower river and water from Greenwood Lake was not superimposed upon main river flood. At Hartford, gage height was 37.56 feet, compared with previous maximum of 31.4. At height of flood, 20 percent of city was under water to depth of up to 25 feet. Power plants of both Springfield and Hartford were put out of commission, but water supplies were not affected, both cities drawing their water from upland sources. **Hudson Crest Reduced by Reservoir.** EDWARD H. SARGENT. 468. Brief data given on flood flows on Hudson River and tributaries. It is estimated that Sacandaga Reservoir reduced flood peak at Albany by approximately 50,000 second-feet, probably lowering flood height about 4 feet. Flood was anticipated owing to deep snow cover and absence of January and February thaws and reservoir was accordingly depleted in preparation for it. Probable 1000-year flood predicted from calculations made at time reservoir was designed was exceeded by nearly 10,000 second-feet. **Utility Damage in New England.** H. S. KNOWLTON. 469. Brief data regarding difficulties experienced at power plants. Damage to Roads and Railroad Lines. 469. Brief

details. **Washington and the Potomac.** I. D. Foos. 469. Flood was worst since 1889 and in places exceeded flood heights of that year. Power, communication, transportation, and water supply services were disrupted on both banks.—*R. E. Thompson.*

Record Floods in Northeastern States. II. Eng. News-Rec., 116: April 2, 1936. **Wheeling Recovering from Its Worst Flood.** H. W. RICHARDSON. 495–6. Lower business district was flooded (Ohio River) to depth of 12 to 15 feet and Wheeling Island, residential section of city, with population exceeding 8,000, was inundated; in some places, to depth of 20 feet. Previous all-time flood record was in 1884, when water reached stage of 52 feet on Warwick Dam gage, 4 miles above city. Flood of 1913 reached stage of 51 feet. This year, flood gage stood at 55.5 for 4 hours. Municipal water supply was not damaged and continued functioning throughout flood. **Norris Dam Lowers Tennessee Flood Crest.** 496. It is estimated that flood crest in Tennessee River was reduced 3 feet at Chattanooga by Norris Dam, saving several hundred thousand dollars. Flood stage at Chattanooga is 30 feet. Crest was 37.1 feet. **Central New York Repeats July's Floods.** S. C. HOLLISTER. 496–7. Heavy spring rains and frozen or saturated ground produced high floods in Central New York. Precipitation was highest ever recorded for March at Ithaca, with total to 8 a.m. March 21 of 6.25 inches, as compared with previous high record of 4.75 in 1896: normal March precipitation is 2.31 inches. Flood crests were higher and damage in many places greatly exceeded that of July flood, which was estimated at nearly $40 per capita over area embracing 16 counties. Flood protection works are badly needed. **Record Floods in Eastern Canada.** ROBERT F. LEGGET. 497–8. In eastern Canada, March floods were worst in history. St. John River, main stream in New Brunswick, always susceptible to floods, rose higher than ever before, inundating about 150 square miles. Water level in Fredericton was 23 feet above normal. **Flooded Water Supplies Safeguarded.** A. F. DAPPERT. 498. Flood levels considerably in excess of those of last July were reached in New York state. Serious damage was confined primarily to communities along main rivers. In Binghamton, considerable area was inundated and flood water reaching pumping station necessitated shutdown of filter plant. Some service was maintained for about 24 hours from storage reservoirs. Acute shortage developed, however, which was met to some extent by utilizing chlorinated well supply of Ansco Company and obtaining about 5 m.g.d. from Johnson City through interconnection between systems. Regular supply was off for about 72 hours. Wells and pumping station at Endicott were flooded and chlorinated emergency supply was developed from deep well in West Endicott. At Norwich, flood water washed out 8-inch and 10-inch mains, necessitating suspension of filter plant operation. Temporary hose lines were strung across bridge and later replaced by emergency pipe line. Unfiltered water was delivered for time, but at all times it was heavily chlorinated. At Granville, regular intake became plugged, necessitating use of emergency intake. All water delivered was adequately chlorinated. At Rensselaer, filter plant was shut down, owing to possible leakage into clear well. Sufficient stored water was available to enable service to be maintained. **New England Supplies.**

H. S. KNOWLTON. 498. Emergency measures outlined. In Lawrence and Lowell, water supplies and pumping stations were submerged; in Methuen, wells and pumps were rendered inoperative; in Newburyport, surface water supply plant was submerged, but ground water supply was saved by construction of sandbag dikes. Lawrence Experiment Station was flooded to depth of 2 feet over laboratory floor. **Water Supplies in Potomac Valley.** R. B. GARABRANT. 498. Hagerstown, Maryland, has 2 sources of supply, mountain gravity system and pumped Potomac River system. Pumping plant, sedimentation basin, filters, etc., of latter are surrounded by continuous dike with top at elevation 376. Crest of March flood was El. 378.7 and entire plant was flooded to depth of 10 feet. It is expected plant will be out of service some weeks. Meanwhile the 4 communities dependent on supply, with normal consumption of 4.5 m.g.d., are being rationed from mountain supply, which was undamaged and has maximum yield of 3.3 m.g.d., with 100 m.g. storage.—*R. E. Thompson.*

Harrisburg Water Supply Restored to Service. Eng. News-Rec., 116: 507, April 2, 1936. Harrisburg, Pennsylvania, supply was out of service 8 days as result of flooding of pumping station and filter plant during March 18–19 flood on Susquehanna River. Plant is again in service, but operation is far from normal. Only 35 m.g. was in storage reservoirs when supply failed. Stringent restrictions reduced demand from 12 to 5 m.g.d. Three private companies neighboring towns aided in keeping reservoirs from being entirely depleted. When pumping was resumed, only 5 m.g. remained in storage. Fleet of trucks hauled water from Lemoyne to reservoirs for several days. Studies are being made to determine if pumping and filter plant should be moved or whether adequate protection against recurring floods is possible at present location.—*R. E. Thompson.*

Mopping up after the New England Floods. E. J. CLEARY. Eng. News-Rec., 116: 566–71, April 16th, 1936. Conditions noted in personal survey described, and pertinent data given. In Lowell, Massachusetts, crest exceeded by 7.5 feet previous record of 1852 and more than 4 of city's total area of 14 square miles was inundated. Preliminary estimates place damage to public works at $2,700,000. When water subsided, 2-foot layer of mud covered streets. Toll of Merrimac River at Manchester, New Hampshire, which rose 17.4 feet above Amoskeag Dam, 8.4 feet above previous record, is estimated at $5,000,000. At Lawrence, Massachusetts, river reached height 10 feet above previous high mark established in 1896 and 1.2 of city's total area of 6.75 square miles was inundated. Critical situation developed when slow sand filters, located on river bank, were flooded. Pumping plant was also flooded and only 40 m.g. of potable water remained in reservoirs. Daily consumption is 5.5 m.g. Emergency connections with near-by towns were made by means of hose lines and fire department pumpers aided in maintaining pressure. More than one-twelfth of Haverhill, on Merrimac, was under water when river reached stage 8 feet above previous high in 1887. Most flood-ravaged community in New England was Hartford, Connecticut, where more than 25 percent of city was inundated, in some places to depth of 30 feet, when Con-

necticut River swept over dikes built above any previous high water level, breaching it in one place for about 1000 feet. Streets were torn out to depth of 12 feet, frame buildings wrecked, scores of steel oil tanks overturned, and entire basin covered with 2-inch layer of gasoline. Water supply, derived from upland sources, remained in operation. Destruction of sprinkler systems in factories caused consumption to increase from 18 to 30 m.g.d. After river level had receded, there was still some 30 feet of water behind dikes that could be released only by cutting channel through dike wall. Flooded Connecticut River entered Springfield by 2 routes: north section is surrounded by dike that was overtopped and central and southern sections, which slope from high river bank, were submerged by water that entered through sewer system. Water supply, from upland source, remained in service, but many mains were broken. More than 6 of 34 square miles comprising city were under water.—*R. E. Thompson.*

Safeguarding Water Supplies in Flooded Massachusetts. Eng. News-Rec., 116: 571–2, April 16, 1936. State Health Department emergency organization and activities are outlined. Before difficulties arose in Lowell, rumor of impending water shortage caused residents to collect reserve supply in bath tubs, etc., with result that normal consumption doubled. Eventually, polluted river water flooded well field and then entered open conduit leading to 30 m.g. storage reservoir located on high ground, normal 5-day reserve supply thus being contaminated. Emergency chlorinators were set up on distribution main from reservoir and high residual chlorine maintained. Meanwhile tubular well supply remote from river, abandoned some years ago owing to high iron content, was placed in service, supplying sufficient water to take care of one-third of normal requirements. Situation in Lawrence was perhaps most critical. Slow sand filters located on river bank were completely inundated and dependence was placed on gravity storage that was rapidly depleted by panic-stricken inhabitants who attempted to store up reserve in every available container. Emergency supplies were obtained from neighboring communities in 2 ways: hydrants were connected by hose and fire pumpers were utilized to relay water through hose from nearby sources and to boost pressure. All water is being doubly chlorinated and residual of 0.2 p.p.m. is being carried at storage reservoir. Historic Lawrence Experiment Station, now the bacteriological laboratory of the State Department, was scene of feverish activity in making emergency water analyses until laboratory was flooded. Technicians worked in hip-boots until water came within 2 inches of top of tables. Samples and equipment were then transferred by boat to high ground, where work was continued under difficulties using hastily constructed incubator built over gas range.—*R. E. Thompson.*

After the Floods in the Pennsylvania Region. H. W. RICHARDSON. Eng. News-Rec., 116: 599–603, April 23, 1936. Hardly a town on banks of Susquehanna River and tributaries, which drain eastern slope of Allegheny Mountains, escaped flood. Water supply failed in only 2 of large cities. Extraordinary measures by state and local health authorities prevented disease epidemics. At Williamsport, crest stage was 1 foot above record of 1889.

Water supply remained intact. At Wilkes-Barre, flood dike was under construction. Most of embankment was carried away and 30 percent of city's area flooded. Water supply was not affected. Crest stage was same as record of 1865. North Branch of Susquehanna River broke through dike at upper end of Sunbury and inundated 80 percent of town. Water supply of Pittsburgh was restored in few days, $40,000 repair contract being completed in 90 hours. During period of shortage, water was hauled in tanks by department of streets.—*R. E. Thompson.*

Cities Without Water. II. Harrisburg's Water Works Failure During the Flood. CHARLES H. KRESSLER and RALPH W. FOSTER. Eng. News-Rec., 117: 15–8, July 2, 1936. Harrisburg flood, which reached stage of 30.33 feet, was highest on record, previous high being 27 feet in 1889. Water works consist of intake, filtration plant, and low-lift station on City Island in Susquehanna River; high-lift pumping station on mainland; and 3 reservoirs with combined capacity of 50 m.g. that feed by gravity into system. Filter plant was enclosed by dike to stage of 30 feet in 1904. High-lift station was protected only to stage of 22 feet. When plants were flooded, reservoirs were practically full, but provided only sufficient water for 4.5 days' normal supply. Complete breakdown of system was prevented by obtaining water from adjoining systems, mostly privately owned, and by hauling water from Lemoyne in fleet of trucks loaned to city. Total of 2.75 m.g. was delivered by latter method, military police patrolling entire route to give trucks right of way. All business places, factories, etc., using large quantities of water, were closed, and restaurants were ordered to limit water supplied to 1 glass per person. To avoid outbreak of typhoid, order was issued to boil all drinking water at least 30 minutes and all residents were requested to be inoculated against typhoid. Details included of emergency arrangements with adjoining water companies and of rehabilitation measures employed.—*R. E. Thompson.*

Concerning Certain Sanitary Aspects of the Flood of 1936, and Lessons Therefrom. C. D. HOWARD. Health (N. H. State Board of Health), April, 1936. Combination of heavy snowfalls during January and February, 1936, with rains during early March, and with extraordinarily heavy precipitation on the 18th, spelled most serious disaster in annals of New Hampshire. Fortunately, relatively few public water supplies were affected in serious degree. Many pumping stations were flooded; at Franklin the filtration plant was out of commission for a number of days because of penetration of flood water from Pemigewasset River. At Plymouth, draft was confined to west side supply during period that Intervale wells were flooded, a chlorinator being installed here with resumption of this source. Emergency chlorinators were also installed at Farmington, Epping, Raymond, and Merrimack, it being necessary at the last-named to truck water for a few days from Reed's Ferry system.—*G. C. Houser.*

Public Water Supplies Affected by Flood Conditions. Health News (N. Y. State Dept. of Health), 13: 12, 47, March 23, 1936. Heavy rains, thaws, and ice jams which occurred during period March 12 to 14 affected several public

water supplies, principally in eastern New York. No serious consequences were experienced. At village of Mohawk, ice jam across creek caused rising waters to back up and flood well used as source of supply. Delhi water supply became so turbid as result of heavy run-off that it was necessary to by-pass filters for several hours. Similar condition occurred at Fort Ann. Well which supplies water to village of Hoosick Falls was flooded by river water.—*G. C. Houser.*

Snow Surveys Predict Runoff. G. A. HATHAWAY. Eng. News-Rec., 116: 728-30, May 21, 1936. System of snow-survey courses was established early in 1933 in mountainous areas of upper Missouri River basin above Fort Peck Dam, now under construction. From data obtained, together with 25-year records of streamflow, precipitation, and temperature, estimate was made in March, 1935, that probable April-July runoff would be 55 percent of normal. Actual recorded flow was 49 percent. With added experience and longer snow-survey records, it is anticipated that reliability of forecasts will increase. Details of procedure employed are included.—*R. E. Thompson.*

DAMS

Nine Hundred Dams Inspected. GEORGE W. HAWLEY. Eng. News-Rec., 117: 71-6, July 16, 1936. For purpose of safeguarding life and property, California State Legislature of 1929 made it mandatory upon state engineer to exercise supervision over all dams in state, other than those federally owned, whether already built, or to be built, which are 25 feet, or more, in height, or which store 50 acre-feet, or more, of water. Act, which was passed after catastrophic failure of St. Francis Dam, decrees:—(1) inspection of existing dams; (2) approval of plans and specifications for, and inspection of construction of, new dams, or enlargement, alteration, repair, or removal of existing dams; and (3) supervision over maintenance and operation of all dams in so far as necessary to safeguard life and property. Three-year period was allowed for examination of all existing dams and thereafter state engineer was required to issue either certificate of approval that dam was safe, or order requiring owner to make such changes (specified in each case) as found necessary. Total number of dams built prior to effective date of Act which were examined critically by department is 830. On these, state has reviewed and passed upon 422 applications and plans for repairs and supervised repair work. Total number of dams passed upon is 950. Plans, specifications, and sites have been reviewed and construction supervised of 110 new dams and 30 enlargements. Dams now under state jurisdiction total 588. Studies made included stress analysis of structure itself, hydrographic investigations dealing with estimated maximum runoff, and hydraulic consideration of spillway capacity, overpour depth, etc. Program of examinations and experiences are discussed. The 830 old dams reviewed fell into 3 general classes, each embracing roughly ⅓ of total: (1) structures that could be approved immediately; (2) those that required more thorough study before final decision; and (3) those that needed alterations at once. Worst faults found, in order of frequency of occurrence, were:—(1) inadequate spillways; (2) inadequate treatment of foundations;

(3) lack of protection against erosion; and (4) poor materials and construction methods.—*R. E. Thompson.*

Hydraulic Fill at Quabbin Dike. Eng. News-Rec., **116**: 882–6, June 18, 1936. Details given of carefully-planned, suggested operations for hydraulic-fill construction prepared by engineers of Metropolitan District Water Supply Commission for Quabbin Dike, first of 2 earth dams that will complete Boston's new Quabbin Reservoir water supply system, and of independent plan adopted by contractor. Latter involves truck haul of dry materials, mechanical feeding of materials to sluice box over rotary grizzly, and dredge pump delivery of mixed material to dam through 2 lines of discharge pipe. Specifications require 2,100,000-cubic-yard embankment to be placed by full hydraulic method, deposition of silt in central pool forming impervious core, held in place by shoulders of coarse, pervious material. Dike, 135 feet high, 800 feet wide at base, and 2140 feet long on crest, will close saddle between 2 hills at rim of proposed reservoir. Thirty-one concrete pneumatic caissons were sunk end to end across valley as tight core wall. In addition to hydraulic fill, dam requires 175,000 cubic yards consolidated pervious foundation embankment, 145,000 cubic yards rolled impervious embankment, 100,000 cubic yards selected coarse material, and 139,000 cubic yards riprap and crushed stone blanket.—*R. E. Thompson.*

Practical Soil Mechanics at Muskingum. THEODORE T. KNAPPEN and ROBERT R. PHILLIPPE. Eng. News-Rec., **116**: 453–5, March 26; 532–5, April 9; 595–8, April 23; 666–9, May 7, 1936. Extended description and discussion of soils-laboratory investigations made in connection with design of Muskingum River Valley flood control dams, which involved adapting soils of widely different characteristics to construction of earth dams on wide range of foundation materials. Second article deals with seepage. As reservoirs are primarily for flood control, allowable seepage depends upon effects of such flow on stability of structure. Details are given of procedure employed and results of permeability coefficient tests and model studies of seepage. Designs were improved and costs reduced by careful application of results of these studies and authors state that it is difficult to overestimate value of permeability model studies in developing design of earth dams. Most valuable result was development of impervious rolled upstream blanket in place of cutoffs at very considerable economy. Model studies showed that, with one exception, cutoff could not be constructed at reasonable cost that would have appreciable effect on seepage. In third article, details are given of foundation-strength design studies. Foundations were of questionable character at sites of several dams and tests were undertaken to determine definitely behaviour to be expected under proposed loadings, including determination of consolidation characteristics of foundation materials and stress-strength relationship in foundations. For purpose of checking analytical application of JURGENSON'S formula and to develop methods for cases not covered by formula, photo-elastic method of determining stress distribution experimentally was devised in laboratory, using gelatin as foundation medium and lead shot as embankment medium. Average deviation between 2 methods was 6 percent

and maximum, 19 percent. Practice has been to design with factor of safety of 1.5 against over-stresses and estimated factor of 2.6 against failure. As foundation strength increases during construction, due to consolidation, desired factor of safety may be obtained in some cases by limiting rate of construction. System of settlement gages has been installed at several dams to check foundation consolidation during construction to ensure that factor of safety is maintained. In final article of series, selection and control of embankment materials is discussed and details given of laboratory organization and costs. Equipment described includes apparatus for taking undisturbed cylinder samples in field and device developed for tamping material so as to simulate action of sheepsfoot roller. Total cost of all laboratory equipment was approximately $9,000 and total operation and construction cost of laboratory to end of design period $32,000. Estimated cost of 14 reservoirs is $40,000,000. Core borings, auger borings, and test pits cost $225,000. The $32,000 + $12,000 for geological studies represents cost of rendering data obtained in explorations practically applicable. Experience shows that to every problem in soil mechanics an answer can be found in laboratory, which, in proper design and construction of earthworks, will pay for its cost many times over. Correction, Eng. News-Rec., **116**: 711, May 14, 1936.—*R. E. Thompson.*

Earthquake-Proof Earth Dams. F. H. TIBBETTS. Eng. News-Rec., **117**: 10–13, July 2, 1936. Details given of design of 5 roll-fill earth dams, 100 to 140 feet high, built for Santa Clara Water Conservation District on principal streams flowing into Santa Clara Valley to store waste flood water which will be released for seepage into underground storage. At 4 of 5 sites there is complete geological non-conformity on opposite sides of streams. Not only are foundation materials deeply weathered, but they are shattered, broken, and of highly variable quality. Largest dam, on Coyote River, actually straddles active Hayward fault on what is probably as poor foundation as ever was selected for large dam and is literally poised over city of 70,000 population. Usual cross-section of dams is 20-foot top width with slopes that average 1 on 2.5. Coyote Dam has top width of 100 feet and slopes ranging from 1 on 3.5 to 1 on 4. Rear portions were constructed of most pervious materials available and front portions of most impervious material within economical hauling distance. Line of demarcation between materials is indefinite. Selection and placement was under careful laboratory control. Following requirements were assumed to be essential for resistance to earthquakes. (1) Impervious or water-resisting element of type that cannot be ruptured by shock. (2) Deep cutoff wall to prevent water passage through cracks under foundation. (3) Dimensions, especially of impervious element, such that under no condition could this portion of structure be offset sufficiently to let water through. As result of thorough study, it was concluded that fault movements might have horizontal component of about 15 feet and vertical component of from 3 to 5 feet. On this basis, minimum thickness of impervious element at top was made 4 times anticipated maximum horizontal movement and freeboard was provided equal to 4 times maximum vertical component. (4) Protection of impervious element by weighting with granular material possessing no shear resistance. (5) Type of construction such that fissures formed by movements

of foundations which stretch dam lengthwise will be instantly and automatically closed before water can establish channel. Essential feature in design of Coyote Dam is location of major bulk of structure on upstream side of rock abutments or constriction of canyon walls which form dam site. If reservoir is full and dam saturated at time of earthquake, gravel and rockfill blankets will hold earth core in place and latter will retain water. In event of stretching endways, gravel with heavy rock on top will fall into cracks and prevent water from rushing through.—*R. E. Thompson.*

Measuring Bed Rock Action under Tygart Dam. J. E. DEIGNAN. Eng. News-Rec., **116**: 731, May 21, 1936. Tygart Dam is straight gravity structure with crest length of 1850 feet and maximum height above rock of about 232 feet, located on Tygart River near Grafton, West Virginia. Spillway section, 490 feet long, will hold reservoir level 23 feet below top of abutment section. Central part of dam is founded on sandstone blanket, 20 feet thick, which is badly seamed and blocked, and which in turn overlies soft stratified shale, containing occasional clay and carboniferous veins. To measure deformations in shale at various depths over base area of typical monolith under various loadings of dam, nine holes from 30 to 170 feet deep, were drilled in shale over base of one of spillway monoliths in which plungers were installed which transmit deformations to specially-constructed electrical elastic-wire deformation gages set in concrete of dam, lead wires being run to common terminal board located in adjacent gallery. Brief details given of equipment and installation.—*R. E. Thompson.*

Design of Bills Brook Dam, Barkhamsted Reservoir. WILLIAM DORENBAUM. Jour. New Eng. W. W. Assoc., **50**: 3, 302–14 (1936). Studies were begun in 1925 to enlarge water supply of Hartford, Conn., supply to come from East Branch of Farmington River by constructing reservoir impounded by Bills Brook Dam. History of development and various preliminary studies are described. Construction was started in 1933. Dam is a rolled earth embankment with a concrete core wall founded on solid rock, located on East Branch of Farmington River just above mouth of Bills Brook, is 1950 ft. long, has max. thickness of 900 ft., max. height above river bed 137 ft., and is 131 ft. thick at flow line. Copy of plans of Bills Brook Dam, cross section of dam and of stream control works are included. Model studies were made of the spillway to study hydraulic features of weir and channel and the matter of erosion at discharge and also of the diversion conduit. Unit and total costs of two contracts covering construction of stream control works and lower portion of dam and for construction of second portion of dam are given in detail.—*Martin E. Flentje.*

Thin Arch Dam of Masonry Meets Park Requirements. Eng. News-Rec., **116**: 725–6, May 21, 1936. Brief description of Barney Williams Dam, constructed for Phoenix, Arizona, in Horse Thief basin recreational area, 50 miles north of city, comprising 4400 acres in high, well-wooded section deeded to city by Congress in 1935. To provide potable water supply, dam was required for developing surface storage, tight rock formation providing little oppor-

tunity for underground storage. Site was located where 60 acre-feet could be stored by dam 57 feet high. As park is inaccessible except by 95-mile road including many steep grades and there was excellent granite quarry site few hundred feet below dam, masonry structure was natural choice, additional advantage being that its attractive appearance would harmonize well with adjoining landscape. Total cost was $32,000. Arizona state engineer's office, division of dams, imposed 3 requirements: no tension, no overhang, and compression limited to 400 pounds per square inch. These were met satisfactorily.—*R. E. Thompson.*

Brick Veneer Holds Tight. Eng. News-Rec., 116: 699, May 14, 1936. Brief illustrated description of condition of Caneadea Creek Dam (constant angle concrete arch) of Rochester Gas and Electric Company, which was faced on both sides with vitrified paving brick when built in 1928 to prevent disintegration of concrete. Brick surface is blotched by leaching from some of construction joints and some expansion joints show surface spalling; but in no place is there structural failure of facing, or loosening of bond between brick and concrete.—*R. E. Thompson.*

Controlling Flashboard Drop by Collapsing Pipe Supports. CLIFFORD A. BETTS. Eng. News-Rec., 116: 627–8, April 30, 1936. Reservoir level regulation between comparatively narrow limits can be facilitated by raising height of dam with flashboards that will collapse under predetermined head and thus by-pass excess flood water. Additional storage capacity conserved by flashboards during dry periods is of considerable economic value in water supply developments, etc., and when cost of obtaining adequate spillway capacity for earth dam by simple overfall spillway is prohibitive, automatic flashboards permit provision for handling rare floods which otherwise might prove disastrous. Assurance that boards will fall is essential to plan. To establish tolerances and determine limits within which collapse will occur, tests of full-size flashboards were recently made at National Hydraulic Laboratory of National Bureau of Standards in cooperation with United States Forest Services of Department of Agriculture. Investigations, details of which are given, were confined to standard galvanized steel and wrought iron pipe, solid steel pins having been previously tested and found more difficult to control for smaller installations. As result of tests, installations have been made with flashboard gates hinged at base so that after by-passing flood they can be re-erected readily and new pipe installed. During recent high water, hinged gate at Sherando Dam, Virginia, 35 feet long and 3.5 feet high, functioned very much as found in experimental work, gently falling in its own surge when water reached elevation within 1.5 inches of height indicated by derived formulas.—*R. E. Thompson.*

Wear on Sheepsfoot Rollers. PAUL BAUMANN. Eng. News-Rec., 116: 835–6, June 11, 1936. Details given of modified design of petrolithic rollers used for compaction of stony soil of San Gabriel Dam No. 1 of Los Angeles County Flood Control District. Despite sharp, rocky material, wear on feet after several months' operation has not been enough to require any replacements and required compaction has been exceeded.—*R. E. Thompson.*

Solving a Dam Break by Deduction. P. E. GREEN. Eng. News-Rec., 116: 556–8, April 16, 1936. Herrin, Illinois, city of about 10,000 population, is served by water supply reservoir on Little Wolf Creek, 12 miles south of city. On night of June 21–2, 1935, after unusually rainy spring, torrential rain occurred. The dam, an earth structure 700 feet long, 40 feet in maximum height, and with puddle corewall, was washed out. Concrete spillway, 100 feet wide, of 4-foot effective depth and located at east end of dam, was not damaged. Accurate information on failure, which was unwitnessed, was important to facilitate reconstruction. Nearest rain gage is 12 miles distant, but data from surrounding country indicate that total fall was not less than 6 inches over period of 6 hours, with probable maximum of between 4 and 5 inches in 2 hours, whereas maximum in 24-hour period during last hundred years would have been about 8 inches. From observations and calculations it is believed that runoff was not less than 1500 and may have been 2400 second-feet, that spillway capacity of 2600 second-feet was sufficient to pass flood, and that dam failed from overtopping or from structural weakness, due to settlement or leakage, or from both. Indication was that dam had settled more than 2 feet at central point and, if this were true, it was evident that when spillway was running about half-full, carrying about 1500 second-feet, water would begin to run over dam, with inevitable effect of destroying it. Corewall at deepest point had not been carried to rock foundation and soil on creek bottom was largely blue clay. It is believed that weight of central part of dam caused subsidence in blue clay and that while corewall was generally tight, channel must have formed through, or below, it which raised line of saturation, caused dam to settle heavily, and resulted in its destruction while spillway was being taxed to but one-half its capacity. Dam is now being rebuilt of same cross-section, but with corewall of graded materials and carried down to rock, and with downstream slope drainage.—*R. E. Thompson.*

Elk City Earth Dam Fails During Deluge. Eng. News-Rec., 116: 678, May 7, 1936. Long, low earth dam forming part of water supply system of Elk City, Oklahoma, failed about 1 a.m. May 1, when spillway was unable to pass cloudburst and structure was overtopped to depth of 18 inches for full length of 2019 feet. At dam, rainfall was 5.75 inches in 2 hours, of which 3.5 inches fell in 45 minutes. Fall of 8 inches during evening on drainage area was indicated. Upstream face of dam, which had slope of 3:1, was covered with concrete slab and 2:1 downstream slope was sodded. Maximum height was 20 feet. When built, in 1925, spillway was about 5 feet deep, providing capacity of 10,000 second-feet. Three years ago, lip was raised 2.5 feet, reducing capacity to 4400 second-feet. At new spillway level, reservoir covered 90 acres and held 600 acre-feet. Reservoir was practically empty before storm. Maximum runoff during storm, from drainage area of 12,500 acres, is estimated at 15,000 to 18,000 second-feet. Rainfall of 0.6 inch would have filled reservoir. In addition to main break, 150-foot section in center, dam was nearly washed through in several other places and whole downstream slope badly eroded. Last year dam was overtopped with slight damage. In 1934, reservoir went dry and city dug 3 wells below dam and 2 in reservoir area. These were flooded out but were soon partially restored, averting serious water shortage. PWA

project ($350,000) is now under way to provide additional supply from wells in Red River valley, 12 miles distant.—*R. E. Thompson.*

WATER SUPPLY—GENERAL

Developments in Water Works Construction. CHARLES B. BURDICK. Civil Eng. 6: 7, 449–53 (1936). One of a group of papers discussing sanitation in the Miss. Valley. This valley and the Mid-South together contain ⅓ of the cities of the U. S. over 100,000 pop. and nearly ⅓ of those exceeding 200,000. Of the 33 cities with over 100,000, 19 use surface water supplies only of which 18 are filtered, 4 filtered and softened and 6 water from impounded reservoirs; 9 cities use ground water sources only and 5 use combined surface and ground sources. In author's review of water treatment it is pointed out that since first U. S. filter was constructed in 1882 at Sommerville, N. J., 1782 filter plants have been built up to 1930, serving 29,000,000 people with 3826 m.g. of water per day. Concrete filter units have grown from ½ and 1 m.g.d. units up to 6.3 m.g.d. units in the new plant at Milwaukee. This probably represents max. economical unit size, which is limited by the pipe connections for purposes of washing. Sketches and data are given of 4 methods of surface sand washing to aid upward flow wash. Progress in taste and odor elimination is discussed. Water softening is practiced in 152 municipal softening plants, majority of which are in Miss. valley. In ordinary filtration plant increased nominal expenditure for softening for equipment and increased operating expense is between 1c and 3c per 1000 g. treated. Results of recent research studies indicate soap savings equal to approx. 72c per capita per year for each 100 p.p.m. of hardness removed.—*Martin E. Flentje.*

To Protect Portland's Water Supply. BEN S. MORROW. Civil Eng. 6: 10, 656–58 (1936). Water supply of Portland, Ore., is obtained from Bull Run River, approx. 30 min. east of city. Stream above diversion has a water shed area of 102 sq. mi. and lies between elevations 750 and 4700, average flow at headworks has been 812 cu. ft. per sec. with max. of 21,000 and low of 64. Stream supply is augmented by 3 billion gallon storage development in Bull Run Lake, head waters of main stream; and 11 billion gallon Bull Run Storage Project located on main stream 5 mi. above headworks. Dam is 950 ft. long and 200 ft. high, of concrete, built in 1928–29. Daily bacteriological tests show water quality to be in excess of requirements for drinking water. Chemical results obtained by U. S. Geological Survey at 10 day periods throughout year average: total solids 30, iron 0.03, bicarbonates (HCO_3) 12, chloride 1.3, total hardness 8.8.—*Martin E. Flentje.*

Colorado Water. Eng. News-Rec., 116: 824, June 4, 1936. Brief details given of United States Supreme Court decision denying petition of State of Arizona for leave to enter suit against 6 other states of Colorado River basin, with view to acquiring permanent title to definite share of Colorado River water, share to be determined by Supreme Court. Act of Congress will be necessary before proposed suit will be entertained by Court.—*R. E. Thompson.*

New Gravity Water Supply for Big Stone Gap, Virginia. W. MARTIN JOHNSON. The American City, 51: 9, 75, September, 1936. By construction, 1500

feet above the town of Big Stone Gap, Va., of a masonry dam, an ideal source of water, which needs only chlorination before consumption, has been provided. The town had long controlled watershed on which dam was built, being thereby enabled to eliminate pollution. Dam is also, owing to its great elevation, a potential source of hydro-electric power.—*Arthur P. Miller.*

New Water Supply for New York City. Eng. News-Rec., 116: 858, June 11, 1936. Appropriation of $17,500,000 has been authorized for sinking shafts on 85 miles of tunnels and preliminary work on 3 reservoirs of Delaware River project, which has been delayed for nearly 15 years by legal and financial obstacles. Total cost of completed project will be about $272,000,000 and construction will take from 7 to 10 years. Present consumption is close to 1000 m.g.d. and available supply only 1039 m.g.d. New supply will provide additional 540 m.g.d. Project contemplates development of 2 tributaries of Delaware, East Branch and Neversink Rivers, and also Rondout Creek, tributary of Hudson River. Water will be delivered through pressure tunnel aqueduct across Hudson Valley to West Branch reservoir of Croton system and thence southerly through Kensico Reservoir to Hill View Reservoir.—*R. E. Thompson.*

Providence Report Favors Extension of Water District. Eng. News-Rec., 117: 224, August 13, 1936. Report recommending extension of Providence Water District to include North Providence, Johnston, and Warwick has been made by Philip J. Holton, Jr., under direction of Charles A. Maguire, Commissioner of Public Works, following study of means of securing increased water consumption. With daily consumption of 27 m.g. from Scituate Reservoir district in 1935, as compared with original estimate of 37 m.g., income falls correspondingly short of anticipation, which is felt as serious handicap. Diversion claims have been paid based on complete diversion by 1955, corresponding to consumption of 61 m.g.d. by that date. Studies have indicated that, even with additional areas included, consumption in 1960 would be only 35.3 m.g.d. Daily reservoir yield in 1935 was 111 m.g. If extension is approved, increase in consumption would be about 2 m.g.d. by 1940, which would provide additional income of $175,000 annually. At present, some income is derived from excess water passing dam through development of power; but income from that source is only 80 cents per m.g., as compared with $240 for domestic use.—*R. E. Thompson.*

Water Works Problems of a Small Municipality. W. D. Stalker. Eng. Contract Record, 50: 571–5, July 1, 1936. Detailed description of water supply system of Simcoe, Ontario, town of 5,500 population. Supply is drawn from filter gallery located about one-half mile west of town in flat between low hill and Kent's Creek, augmented by 2 gravel wall wells. Hard pan, or shale, formation downstream from gallery site holds back ground water in area to some extent. Gallery consists of line of open-jointed perforated vitrified pipe, 15 inches in diameter and surrounded with 16 inches of gravel, laid at average depth of 10 feet below ground surface. Some difficulty has been experienced due to sand and roots entering system. Water from one of wells

contains objectionable amount of iron and plant is being installed to remove it by aëration and base exchange, which will also effect partial softening. Plant will be entirely under pressure and re-pumping will not be required. Bacteriological quality of supply has always been satisfactory and chlorination is not necessary. Average consumption in 1935 was 383,000 g.p.d., or 72 gallons per capita. System is practically 100 percent metered. Water is delivered into system at cost of 4.5¢ per 1,000 gallons, which is 57 percent of total operating cost, exclusive of debt charges. Pumps are driven by natural gas engines, electrically-driven pumps being available in case of emergency. Small electrically-driven pump maintains full standpipe during night.—*R. E. Thompson.*

The Water Supply of a State Institution. BAYARD F. SNOW. Jour. New Eng. W. W. Assoc. **50**, (1936) 197–206. Water supply of a state prison colony at Norfolk, Mass. is described. 10 tubular wells supplied this institution from 1930 to 1935 when rapid growth necessitated enlargement of supply. In design factors institutional supplies resemble requirements of town of similar size, with following exceptions: Institutional habits are more regular with consequent greater variations in peak loads, there exists a tendency to a more definitely planned growth of population and industry, and with regular peak demands much more pronounced from the ave. consumption than in municipal supplies. To meet personal and industrial needs as well as possible heavy drafts in case of fire or riot 2 elevated tanks were provided, each with capacity of 150,000 g. To meet additional supply and storage needs a large covered dug well and 4 additional tubular wells were also installed, supply now consistin gof 14–2½ in. tubular wells 25–35 ft. deep and a large dug well. A number of studies of draw downs, well capacities, etc. are described. It was found the dug well not only increased yield of field but also took the place of additional storage, permitting pumping for short periods at rates much in excess of safe yields from the ground. Article is illustrated by a number of curves including recovery-yield curve, well rating curve and flow graph showing results of pumping entire field and recovery of water levels.—*Martin E. Flentje.*

The Water System of Concord, N. H. PERCY R. SANDERS. Jour. New Eng. W. W. Assoc. **50**, 3, 331–33 (1936). A historical resume of the water supply of Concord. At present total consumption averages 2.7 m.g.d., 55 percent of which is fed by gravity; total miles of mains equals 88.86, number of services 4,869 with 721 fire hydrants. Supply obtained from Penacook Lake and drilled wells in Soucook River valley.—*Martin E. Flentje.*

Supreme Court Enjoins Wasteful Water Use. Eng. News-Rec., **117**: 88, July 16, 1936. Recognized rights of state to specified share in water of interstate stream are upheld and enforced by decision of United States Supreme Court on June 1 in complaint of Wyoming *vs.* Colorado growing out of famous litigation of some years ago. In earlier decision, respective claims of appropriators of flow of Laramie River, tributary of North Platte, were decided on

basis of priority; prior appropriators in Wyoming were held to have rights superior to those of subsequent appropriators in Colorado. Decision was based on doctrine that, since both states follow appropriation principle in their own water adjudication, same principle is controlling in their relations to each other. Subsequently, various diversions were made in Colorado, with consent of state authorities, that violated decreed division of rights. Case decided June 1 was brought by Wyoming to enjoin these diversions. Decision recognizes right of Colorado appropriators to transfer, exchange or sell water rights between themselves, so long as total decreed to Colorado is not exceeded and exchange is permitted and recognized by state authorities. Decision introduces important new doctrine in respect to wasteful use of water. Earlier decree fixed particular diversion at 4,250 acre-feet per annum for irrigation of 4,250 acres, but actual diversions in recent years have ranged from 36,000 to 62,000 acre-feet per year, it was complained. Colorado's defense was that return flow reduced actual consumptive use to amount fixed in decree. Court believes that expression "divert and take" used in earlier decision refers to water taken from stream at point of diversion and not to variable and uncertain part of it that is consumptively used. As it was plainly shown that diversions are being made largely in excess of amount fixed in decree, court thinks injunction should issue forbidding further departures from decree in this regard. Wyoming also complained that its representatives were not permitted to measure water at points of diversion in Colorado. Court states, "While the problem of measuring and recording the diversions is a difficult one, we entertain the hope that the two states will by cooperative efforts accomplish a satisfactory solution of it. . . . Wyoming will be given leave to appeal later on for an appropriate order respecting the measurement and recording of all diversions in event the two states are unable to agree."—*R. E. Thompson.*

Modern Water Supply. O. LÜSCHER. Schweiz. Verein von Gas- und Wasserfachmännern, Monatsbulletin, **16**: 7/8, 147, July 1936. Perspective is given of water supply conditions peculiar to Switzerland. Due to abundance of water, in most cases supply of primary (i.e. in its natural state) permanently safe water may be secured; hence the number of purification plants, especially for smaller installations, is extremely limited. Statistics of water consumption for 1934 from 68 largest municipalities indicate a minimum per capita consumption per day of 53 gallons in only two places and an average consumption of 90 gallons; in two-thirds of municipalities investigated, an average consumption of from 106 to 132 gallons, with peak demands over 158 gallons in health resorts, etc. is reached. In city of Zurich for 1929 an average consumption of 75 gallons and maximum consumption of 128 gallons were noted. Total consumption of 54,000 m.g. water was as to 27.5 percent derived from underground sources, as to 25.3 percent from lakes, and as to 47.2 percent from springs; river water being scarcely used. Water from the greater lakes of Geneva, Constance, and Neuenburg is drawn upon extensively and without purification for drinking supplies, lack of soil filtration being compensated for by thorough clarification, immense dilution, and destruction of organic matter. Water from lakes is taken at a depth of approximately 33 feet, yearly fluctuations of temperature being restricted to from 3 to 6 degrees. To eliminate

periodical growth of algae and recently of oscillatoria, slow sand filters of the city of Zurich are now equipped with primary roughing filters with 6- to 8-inch layer of fine sand, operated at rate of 58 m.g.a.d. Rapid sand filters both of gravity and of pressure types, with fine sand layers of 0.25–1.0 mm. size, operated at rates up to 288 m.g.a.d., are frequently used for purification of moderately polluted lake water, where small residual suspended matter is permissible and chlorination has been adopted as additional safety measure.—*Manz.*

PUMPS AND POWER

Diesel Engine Maintenance, Operating, and Outage Data. LEE SCHNEITTER. Mech. Eng., 59, 2, 83–88 (1937). Author comments on the lack of collected data on Diesel engine operation and maintenance. Material presented is the result of research work directed toward certain operating problems. (1) **Cylinder Liner Wear as Related to Fuel Used:** Cylinder liners are the largest and most expensive wear item in Diesel engines. Wear is caused by unsuitable material for rings and liners, incorrect design of rings, liners and pistons, excessive pressure between ring and liner, inadequate lubrication, excessive temperatures or chemical action causing corrosion, abrasive matter in the air, and abrasive matter in the fuel. The last item, wear from abrasive in the fuel, is beyond the engine manufacturer's control. A diagram presents the author's data on the relation of liner wear to the ash content of the fuel oil, showing that 0.5 percent is the maximum ash content for reasonable liner maintenance costs. (2) **Effect of Wear on Diesel Capacity:** Data are presented to show that normal wear in a Diesel engine will reduce the maximum capacity from 15 to 20 percent by the time that the engine is ready to overhaul. The author warns that this loss of capacity must be allowed for in selecting the size of an engine to handle a given load. (3) **Effect of Wear on Diesel Engine Efficiency:** The same wear that reduces capacity reduces efficiency. The fuel consumption at the time an engine overhaul is needed is shown to have increased 22 percent, or an average of 11 percent increase for the entire operating period. Evaluation of the economics of proposed installations should recognize this loss of efficiency. (4) **Diesel Plant Labor Requirements:** A chart is presented to show the average man hours of plant labor that are needed for both operation and maintenance of various size plants. This chart shows that the maintenance labor becomes a higher percentage of the total labor as the plant size increases. (5) **Diesel Plant Maintenance Data:** Data on Diesel plant maintenance is confused and unreliable because of wide variation in plant conditions and reporting methods. Charts are presented to show the cyclic nature of maintenance cost. Costs rise gradually to a peak occurring at four to seven year intervals. The peak cost is when cylinder liners are renewed. These curves show average maintenance costs to be 2 to 3 mills per net KWHr, with peak costs of 3 to 5 mills. (6) **Diesel Plant Outage Data:** These data are from the reports of the Prime Movers Committee, Edison Electric Institute, for 1934, 1935, and 1936. An analysis shows an average service demand availability of 91.7 percent. 66.4 percent of outages were for engine repairs, 28.7 percent for inspection, and 4.9 percent

for generators, auxiliaries, and miscellaneous. A detailed analysis of the specific causes of outages is presented. Crankshaft failures occurring on only 5 of 113 engines reporting accounted for 11.0 percent of the outage. Repairs to pistons and rings, occurring on 71 of 113 engines accounted for 10.1 percent of the outage. The rest of the outage for engines was distributed through 28 other items.—*Homer Rupard.*

Diesel Engines in the Water Works Field. H. L. WITTEK. Eng. Contract Record, 50: 393–6, May 6, 1936. General discussion, including depreciation, normal wear, causes of premature wear, and waste heat reclamation. Diesel-engine drive is particularly adapted to automatic operation. About 1 hour's attention each week is all that is required. Full power is delivered immediately and engines will not stall if load is applied without warming up. Exhaust gases are practically free from carbon monoxide and contain high and constant percentage of carbon dioxide and hence may be used for recarbonation, no taste, odor, or color being imparted to water which cannot be removed by standard methods.—*R. E. Thompson.*

Remodeled Pumping Station and New Diesel Engine, Pennichuck Water Works, Nashua, N. H. WILLIAM F. SULLIVAN. Jour. New Eng. W. W. Assoc. 50, 3, 283–90 (1936). During past 40 yrs. ratio of excess pumping capacity of Pennichunk Water Works to necessary capacity has been about 5, 6, and 7 to 1. 80–90 percent of water is pumped by water power and to provide for emergency possibilities which would have required use of water driven pumps during times of low water, diesel-driven centrifugal pump was installed. Engine installed is a 5 cylinder 4 cycle diesel engine of direct-injection type, having a cylinder bore of 10¾ in. and stroke of 14½ in., normally rated at 250 H.P. at 400 r.p.m. Pump is a single stage double suction centrifugal pump rated to deliver 4200 g.p.m. at 160 ft. total dynamic head with efficiency of 86 percent. On test pump delivered approx. 12 percent in excess of guaranteed capacity at 4 percent less than anticipated fuel consumption. Fuel used will be 41–42 g. per m.g. water pumped at cost of $2.25 per m.g.—fuel oil at 5½¢ per gallon, and pumping against 160 ft. total head. Entire cost including equipment, building, engineering, etc. was $54,260.—*Martin E. Flentje.*

Modernization of Pumping Equipment at Peterborough, Ontario. Eng. Cont. Rec., 50: 179–82, March 4, 1936. Detailed, illustrated description of new 4.5-m.g.d., horizontal shaft, double suction, single stage, centrifugal pump recently installed at Peterborough, replacing old, inefficient, vertical, 3-stage turbine pump. Existing hydraulic turbine was considered satisfactory for continued service, which involved use of right-angle connection. This was accomplished by means of 2-stage, helical and bevel gear, speed-increasing (76 to 1200 r.p.m.) unit, mounted on floor directly over turbine flume. Selected speed is suitable for further installation of 60-cycle synchronous motor, coupled direct to outboard end of pump shaft, as alternative drive. Results of tests shown graphically.—*R. E. Thompson.*